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Optimum Reservoir Operation for Flood Control and Conservation Purposes

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RESEARCH PROJECT COMPLETION REPORT

OPTIMUM RESERVOIR OPERATION
FOR
FLOOD CONTROL AND CONSERVATION PURPOSES

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INTRODUCTION

Statement of the Problem

Rapid population and economic growth in Texas is accompanied by increased needs for water supply and flood control. Depleting groundwater reserves are resulting in an increased reliance on surface water. The rising cost of fossil fuel during the 1970's has focused attention on increasing hydroelectric power generation. Instream flow needs for fish and wildlife habitat and maintenance of fresh water inflows to bays and estuaries have received increased attention in recent years. The climate of the state is characterized by extremes of floods and droughts. Reservoirs are necessary to control and utilize the highly variable streamflow. Due to a number of economic, environmental, institutional, and political factors, construction of additional new reservoir projects is much more difficult now than in the past. Consequently, optimizing the beneficial use of existing reservoirs is becoming increasingly more important.

In addition to ever increasing water related needs, other factors affecting reservoir operation change over time as well. Watershed and flood plain conditions are dynamic. Construction of numerous small flood retarding dams by the Soil Conservation Service and other entities in the watersheds of major reservoirs have reduced flood inflows to the reservoirs. Construction of numerous small ponds for recreation or watering livestock have also decreased reservoir inflows and yields. Increased runoff caused by watershed urbanization is significantly contributing to flooding problems in certain locations. The existing flood control reservoirs were planned and designed based on the expectation of ever increasing intensification of flood plain land use. However, the National Flood Insurance Program has resulted in zoning and regulation of 100-year flood plains. With stringent flood plain management, susceptibility to flooding could actually decrease over time as existing activities choose to leave the flood plain and regulation prevents other activities from moving into the flood plain. Reservoir sedimentation reduces available storage capacity. Construction of additional reservoirs, as well as other related types of projects such as conveyance facilities, flood control levees and channel improvements, and electric power plants, affect the operation of existing reservoirs. Technological advancements in hydrologic data collection, streamflow forecasting, system modeling and analysis, and computer technology provide opportunities for refining operating policies.

Reservoir storage capacities and operating policies are generally established prior to construction and tend to remain constant thereafter. However, public needs and objectives and numerous factors affecting reservoir effectiveness significantly change over time. The increasing necessity to use limited storage capacity as effectively as possible warrants periodic reevaluations of operating policies. Operating procedures should be responsive to changing needs and conditions.

Reallocation of storage capacity between flood control and conservation purposes represents one general strategy for modifying operating policies in response to changing needs and conditions. Reservoir operation is based upon the conflicting objectives of maximizing the amount of water available for conservation purposes and maximizing the amount of empty space available for storing flood waters. Conservation purposes include municipal, industrial, and agricultural water supply, hydroelectric power, recreation, and instream flow maintenance. Common practice is to operate a reservoir only for conservation purposes or only for flood control or to designate a certain reservoir volume, or pool, for conservation purposes and a separate pool for flood control. The conservation and flood control pools in a multiple purpose reservoir are fixed by a designated top of conservation (bottom of flood control) pool elevation. Planning, design, and operating problems associated with flood control are handled separately from those associated with conservation. Institutional arrangements are also based on separating flood control and conservation purposes. Increasing needs for providing water for various uses and reducing flood damages necessitate that limited reservoir storage capacity be used as beneficially as possible. Consequently, consideration of the interactions and tradeoffs between conservation and flood control operations is becoming increasingly more important.

The traditional analysis methods and practices followed in planning and design of reservoir projects and during real-time operations have not really addressed the tradeoffs and interactions between flood control and conservation purposes. In general, expanded analysis capabilities are needed for periodically reevaluating the operating policies of existing reservoir systems. A particular need in this regard is for improved methods for evaluating the tradeoffs involved in reallocating storage capacity between flood control and conservation purposes.

Research Objectives

The research reported herein was directed toward the development of improved management strategies and analysis capabilities for optimizing multiple purpose operations of existing reservoirs in Texas. The work focused largely upon reallocation of storage capacity between flood control and conservation purposes as a management strategy for increasing the beneficial use of limited reservoir capacity. The primary objectives of the investigation were as follows:

- To examine practices and procedures followed in operating reservoirs in Texas,
- To assess the state-of-the-art of reservoir system analysis capabilities,
- To evaluate the potential for increasing the overall benefits provided by existing reservoirs in Texas through improved or updated operating policies, with a particular focus on reallocating storage capacity between flood control and conservation purposes, and
- To develop expanded analysis capabilities for determining optimum reservoir operating policies, with a particular focus on evaluating reallocations between flood control and conservation purposes.

Scope and Organization

The research project on optimum reservoir operation for flood control and conservation purposes was conducted during the period from September 1983 through August 1985. This completion report provides an overview of the study. It summarizes and integrates the findings documented in the reports cited below, as well as providing additional information and views. The overall investigation included several component studies which are documented by two technical reports, a master of science thesis, and a doctor of philosophy dissertation as follows.

Ralph A. Wurbs, "Reservoir Operation in Texas", Texas Water Resources Institute, Technical Report 135, June 1985.

Ralph A. Wurbs, Michael N. Tibbets, L. Moris Cabezas, and Lonnie C. Roy, "State-of-the-Art Review and Annotated Bibliography of Systems Analysis Techniques Applied to Reservoir Operation", Texas Water Resources Institute, Technical Report 136, June 1985.

L. Moris Cabezas, "A Risk-Analysis Based Strategy for Multipurpose Reservoir Systems Operations", Texas A&M University, Ph.D. Dissertation, Fall 1985.

Michael N. Tibbets, "Feasibility of Seasonal Multipurpose Reservoir Operation in Texas", Texas A&M University, MS Thesis, Draft, Fall 1985.

The report entitled "Reservoir Operation in Texas" provides a comprehensive, overview description of how reservoirs are operated in Texas. The report includes: a detailed inventory of the major reservoirs in the state; description of the general environment for reservoir operation, in terms of the water resource and the demands that people place upon the resource; examination of the institutional framework for reservoir management including organizations, programs, and water rights considerations; overview of reservoir operations in each major river basin; analysis of reservoir operation practices and procedures; and discussion of issues, problem areas, and future directions in reservoir operation in the state.

The second technical report provides an assessment of the state-of-the-art of simulation, optimization, and associated stochastic analysis methods applied to reservoir operation as well as a bibliography listing about 800 references, with abstracts for most.

The dissertation by Cabezas consists of developing a procedure for hydrologic and economic evaluation of plans for reallocating storage capacity between flood control and municipal and industrial water supply. The research focused on quantifying the risks and consequences of failing to meet various levels of water demand and failing to provide various levels of flood protection. The methodology was applied to the Waco Lake case study.

The thesis by Tibbets addresses the feasibility of seasonal reallocations of storage capacity between flood control and municipal and industrial water supply. Seasonal variations in various factors affecting reservoir operations were studied. Waco Lake was used as a case study to analyze the hydrologic performance of seasonal rule curve operating policies.

The research documented in detail by the dissertation by Cabezas and thesis by Tibbets and summarized here focused on optimizing the beneficial use of limited reservoir storage capacity in Texas through reallocations between flood control and conservation purposes. However, the findings are also pertinent to evaluating other management strategies for improving operation of existing reservoir systems and to locations other than Texas.

The research addressed the overall process of evaluating proposed storage reallocation plans, which involved a variety of hydrologic and economic modeling tasks. The project was directed toward evaluating, selecting, adapting, and integrating existing models rather than developing new models. General strategies for evaluating storage reallocation plans were developed which incorporated existing analysis capabilities. The various component analyses were performed using manual computations and the following computer programs: MIT River Basin Simulation Model, HEC-5 Simulation of Flood Control and Conservation Systems, and HEC-4 Monthly Streamflow Generation. Analysis strategies were developed and tested using Waco Reservoir in the Brazos River Basin as a case study.

The research and consequently this report are relatively broad in scope. Reservoir operation in Texas and state-of-the-art modelling capabilities were investigated from a broad perspective in order to support development of improved multipurpose reservoir operation strategies and analysis methods in general. The research then focused in detail on one selected management strategy for improving the beneficial use of reservoirs, namely reallocation of storage capacity between flood control and water supply. Chapter 2 of this completion report addresses reservoir operation in Texas from a relatively broad perspective. Chapter 3 is a general discussion of reallocation of storage capacity between flood control and conservation purposes as a strategy for optimizing multiple purpose operations of existing reservoirs under changing conditions. Chapter 4 provides an overview assessment of the state-of-the-art of reservoir systems analysis and describes the models which were selected for use in this study. The procedure developed for evaluating storage reallocation plans is outlined in Chapter 5. The results of applying the procedure to a case study are presented in Chapter 6. Chapter 7 addresses seasonal rule curve operations. The summary and conclusions are presented as the last chapter.

CHAPTER 2
RESERVOIR OPERATION IN TEXAS

Surface water management in Texas is facilitated by 182 major reservoirs with storage capacities greater than 5,000 acre-feet (Wurbs, 1985). Five additional reservoir projects are presently under construction. The 187 major reservoirs contain conservation, flood control, and total capacities of 40.0 million, 18.5 million, and 58.5 million acre-feet, respectively. Streamflow in the state's 15 major river basins and eight coastal basins is highly variable and subject to extremes of floods and droughts. Consequently, the major reservoirs are essential for controlling and utilizing the surface water resource.

Institutional Framework for Reservoir Management

Reservoir development and management is accomplished within a complex system of organizations, laws, and traditions. The water management community consists of water users, flood plain occupants, tax payers, concerned citizens, public officials, environmental organizations, special interest groups, universities, consulting firms, professional organizations, businesses, industries, utilities, and municipal, county, state, federal, and international agencies. Water policy is formulated and management decisions made within a framework of legal and political systems. Water is a publicly-owned resource, and its allocation and use is governed by law.

Reservoir Managers

Within this complex institutional framework, a number of entities are primarily responsible for the actual reservoir operations. The 187 major reservoirs in the state are owned, maintained, and operated by four federal agencies, 43 water districts and river authorities, 39 cities, two counties, a state agency, and 22 private companies (Wurbs, 1985). Table 1 shows the number of reservoirs and storage capacity owned by various types of entities.

The reservoir management agencies can be categorized as federal agencies, state and local governmental entities, and private companies. Most of the major reservoirs in Texas were constructed by state and local governmental agencies or private industry for conservation purposes. However, two-thirds of the total storage capacity is contained in reservoirs constructed by federal agencies. Most of the federal reservoirs are large multiple purpose projects.

Table 1
Types of Reservoir Owners

Type of Owner	Number of Reservoirs	Storage Capacity (acre-feet) Conservation	Flood Control	Total
Federal Agencies	36	17,358,240	16,518,120	33,876,360
International Boundary and Water Commission	(2)	(5,772,600)	(2,654,000)	(8,426,600)
Corps of Engineers	(32)	(11,559,490)	(13,864,120)	(25,423,610)
Other	(2)	(26,150)	---	(26,150)
Water Districts and River Authorities	57	16,080,060	1,324,600	17,404,660
Jointly Owned by Cities and Water Districts or River Authorities	4	2,539,490	248,300	2,787,790
Cities	48	2,843,470	467,000	3,310,470
Counties	5	54,810	---	54,810
Other State Agencies	1	5,420	---	5,420
Private Companies	36	1,093,060	---	1,093,060
Totals	187	39,974,550	18,558,020	58,532,570

Federal Role

Federal agencies have constructed 38 major reservoirs and significantly modified two others. Four additional projects are presently under construction. The federal government is responsible for construction of eight of the ten largest and 21 of the 28 reservoirs with capacities exceeding 500,000 acre-feet. Eight federally-constructed projects have been turned over to nonfederal entities for operation and maintenance. The others are operated by federal agencies. The 43 projects with federal involvement contain 52 percent, 99.9 percent, and 67 percent of the conservation, flood control, and total capacities, respectively, of the 187 major reservoirs. Federal involvement in reservoir construction and operation in Texas is summarized in Table 2.

The five projects constructed by the Bureau of Reclamation were turned over to local sponsors for maintenance and operation. The Bureau of Reclamation continues to own the projects until the local sponsor has completed payments to the federal government for reimburseable costs. The Soil Conservation Service (SCS) has also constructed two major water supply reservoirs which are owned, operated, and maintained by nonfederal sponsors. About 1900 flood retarding dams constructed by the SCS in Texas are not included in the data presented here because controlled storage capacities are less than 5,000 acre-feet for each dam. The Corps of Engineers operates and maintains its projects upon completion of construction. Withdrawals or releases from conservation storage are made at the discretion of the nonfederal sponsors. The International Falcon and Amistad Reservoirs on the Rio Grande River are owned and operated jointly by the United States and Mexico sections of the International Boundary and Water Commission. The Texas Department of Water Resources is responsible for administering the water allocation system and specifying releases from the United States share of the conservation storage.

Institutional arrangements for developing and managing reservoirs are based on project purposes. Practically all reservoirs in Texas containing controlled flood control storage were constructed and are operated by the federal agencies. Almost all the federal projects include flood control. The federal government has borne essentially all costs associated with flood control. The Corps of Engineers is responsible for flood control operations of its own reservoirs and those constructed by the Bureau of Reclamation. The International Boundary and Water Commission handles the flood control operations of its projects.

Table 2
Federal Involvement in Reservoir Development and Management

Federal Involvement	Number of Reservoirs	Storage Capacity (acre-feet)		
		Conservation	Flood Control	Total
Constructed, Owned, and Operated by International Boundary and Water Commission	2	5,772,600	2,654,000	8,426,600
Constructed, Owned, and Operated by Corps of Engineers	27	10,081,790	13,344,820	23,426,610
Presently Under Construction by Corps of Engineers	4	1,348,700	519,300	1,868,000
Major Modification by Corps of Engineers	2	448,600	248,300	696,900
Constructed by Bureau of Reclamation and Maintained and Operated by Nonfederal Sponsors	5	3,081,100	1,779,000	4,860,100
Constructed by Soil Conservation Service and Maintained and Operated by Nonfederal Sponsors	2	17,850	---	17,850
Constructed by Soil Conservation Service and Owned and Operated by U.S. Fish and Wildlife Service	1	18,150	---	18,150
Constructed, Owned, and Operated by Forest Service	1	8,000	---	8,000
Total	44	20,776,790	18,545,420	39,322,210

This data does not include federal grants and loans, such as those provided by the early Works Progress Administration (WPA) Program, which helped finance several of the nonfederal projects.

Municipal and industrial water supply has traditionally been a local responsibility, with the federal government confining itself to a secondary role. However, municipal and industrial water supply storage is included in all but two of the federal reservoirs in Texas, subject to nonfederal cost sharing.

The conservation storage in several of the federal reservoirs is used for irrigation as well as municipal and industrial water supply. However, the Bureau of Reclamation has not constructed large federally-subsidized reservoirs devoted primarily to irrigation in Texas like it has in several other western states. In general, nonfederal sponsorship of conservation storage in federal reservoirs has been handled similarly for irrigation and municipal and industrial uses.

The Southwestern Power Administration (SWPA) markets the power from the three Corps of Engineers hydroelectric power projects. The Western Area Power Administration (WAPA) markets the power from the two International Boundary Commission projects. The SWPA and WAPA sell the power to electric cooperatives, municipalities, and utility companies.

The federal projects all include public access and recreational facilities. Prior to 1965, recreation was included in federal projects as a fully federal expense. The Federal Water Recreation Act of 1965 established development of the full recreational potential at federal projects as a full project purpose subject to nonfederal cost sharing. Recreation contracts have been executed for two projects, which are both presently under construction, under the provisions of this act.

Nonfederal Reservoirs

State and local governmental entities have constructed 108 major reservoirs and one other is presently under construction. These reservoirs contain 45 percent, 0.1 percent, and 31 percent, respectively, of the conservation, flood control, and total capacities of the 187 major reservoirs (Wurbs, 1985). This does not include the seven projects constructed by the Bureau of Reclamation and Soil Conservation Service which are operated and maintained by nonfederal sponsors. Nonfederal sponsors also control all the water supply storage in the Corps of Engineers reservoirs and are reimbursing all costs allocated to water supply.

River authorities own a number of the nonfederal reservoirs and have contracted for the conservation storage in many of the Corps of Engineers projects. River authorities are a special type of water district created to develop and manage water resources from a basinwide perspective. Some river basins in Texas are served by a single river authority while other basins are served by several authorities. The Brazos River Authority, created in 1929, was the first authority ever set up in the United States to administer the waters of a major river. Thus, Texas created its first river authority four years before the creation of the Tennessee Valley Authority by the federal government. The 19 river authorities finance their activities primarily through operation and service fees.

Private companies constructed, own, and operate 36 major reservoirs containing no flood control and less than three percent of the total conservation storage of the major reservoirs. Most of these projects were constructed by electric companies to provide cooling water for steam-electric generating plants.

Water Rights

Texas water law recognizes claims to surface water rights granted under Spanish, Mexican, English, Republic of Texas, and United States as well as Texas state laws (TDWR, 1984). Both the appropriative and riparian doctrines are recognized. The complications of having various forms of riparian and appropriative water rights existing on the same stream has been a significant difficulty in managing the surface water resources of the state. As late as 1968, no single state agency had a record of the number of riparian water users in any major river basin, the extent of their claims, or the amount of water they were using (McNeeley and Lacewell, 1977).

The Water Rights Adjudication Act was passed in 1967 to remedy the confused surface water rights situation. The act required a recording of all claims for water rights which were not already recorded, limited the exercise of those claims to actual use, and provided for the adjudication and administration of water rights. The adjudication process was essentially complete in 1985, with a number of appeals still being undecided.

Under the present procedures, water use permits are obtained through a formal application submitted to the Texas Water Commission. A water use application is approved only if unappropriated water is available, a beneficial use of the water is contemplated, existing water rights are not impaired, and it is not detrimental to the public welfare. After approval of an application, the Water Commission issues a permit giving the applicant the right to use a stated amount of water in a prescribed manner (TDWR, 1984).

In regard to groundwater, Texas courts have followed unequivocally the common law rule that the landowner has a right to take for use or sale all the water he can capture from beneath his land. The state has little control over the use of groundwater. Consequently, conjunctive management of ground and surface waters is extremely difficult.

Interstate Compacts

Texas participates in five interstate river compacts which have been ratified by the states involved and the U.S. Congress. The compacts are administered by commissions representing the member states. The compacts and the dates they became effective are Rio Grande (1939), Pecos (1948), Canadian (1952), Sabine (1954), and Red (1980). The purposes of the interstate compacts are generally to provide for an equitable apportionment between the states of the water available from the river and to facilitate cooperative planning, implementation, and management of projects for the conservation and utilization of the water resource.

Flood Control Versus Conservation Purposes

Reservoir operation is based on the conflicting objectives of maximizing the amount of water available for conservation purposes and maximizing the amount of empty space available for storing flood waters to reduce downstream damages. Each of the major reservoirs in Texas is operated for only conservation purposes or only flood control or a certain reservoir volume, or pool, is designated for conservation purposes and a separate pool for flood control. The pools are separated by a designated top of conservation pool elevation. Institutional arrangements for constructing and operating reservoirs are based on having separate pools for flood control and conservation. Planning, design, and operational problems associated with flood control are handled separately from those associated with conservation.

Construction of a conservation reservoir can actually worsen downstream flood conditions due to loss of valley storage, decrease in flood wave attenuation, and increase in travel time. However, conservation capacity provides some incidental flood protection whenever the flood event coincides with a partially drawn-down pool. Drought periods in Texas have often been ended by a major flood event such that empty conservation storage space was available to store the flood waters. Surcharge storage in conservation only reservoirs may also provide some incidental flood protection. Likewise, temporary storage of flood water in flood control pools may provide some incidental benefits for conservation purposes, particularly hydroelectric power generation. However, reservoir operation throughout the state is based on treating flood control and conservation capacities as distinctly separate pools serving different purposes.

The following discussion is consistent with traditional reservoir planning and operation practice in that conservation and flood control operating procedures are covered in separate sections.

Operation for Conservation Purposes

The world has an adequate amount of precipitation to meet all water use needs for the foreseeable future. The problem is not the total long-term amount of precipitation worldwide, but rather temporal and spatial distribution. The precipitation does not occur at the optimal times and places to meet human needs. Excessive amounts of precipitation flow back to the ocean, and may even cause damaging floods, at some locations and times while severe shortages of water occur at other times and places. The purpose of reservoirs is to alter the temporal and spatial distribution of the runoff resulting from precipitation to better conform to the needs of society. Reservoirs are much more effective at altering temporal than spatial runoff distribution. However, combined with conveyance facilities, reservoirs also are used to transport runoff from one basin to another where it is needed. These general concepts are well illustrated in Texas. Extreme variations in series of wet years and dry years and floods and droughts characterize the Texas climate. The state is also characterized by geographical variations in water availability and water use.

In considering reservoir operation in the state, it is also important to realize that water storage is a regional or local as well as statewide problem. A small region of the state may be experiencing drought conditions while the state as a whole is having a relatively wet year. Physical and institutional constraints often prevent transport of water from a surface water system with a surplus supply to a neighboring system experiencing a temporary severe water shortage. Each local and regional water supplier must have the capability to assure its water users an adequate supply during drought conditions in its own area regardless of the statewide situation. A particular entity is in trouble if its reservoir storage capacity is depleted, even if the combined storage capacity in all the reservoirs statewide is almost full of water. However, on the other hand, complex institutional and physical interactions between localities and regions of the state make surface water management a statewide as well as local and regional problem.

In general, reservoir operation worldwide can be categorized as being primarily influenced by either seasonal fluctuations in streamflow and/or water use or long-term threat of drought. In many parts of the world, a reservoir will be filled during a distinct season of high rainfall or snow melt and emptied during a dry season with high water demands. Thus, the reservoir level fluctuates greatly each year in a predictable seasonal cycle. This is not the case in Texas. Surface water management is greatly influenced by a long-term threat of drought. Water must be stored through many wet years to be available during drought conditions. Although reservoir storage may be significantly depleted within several months, severe drought conditions are characterized as a series of several dry years rather than the dry season of a single year.

Water Use and Availability

The Texas Department of Water Resources (TDWR) recently conducted a comprehensive analysis of the water-related problems and needs of the state in conjunction with updating the Texas Water Plan. The water use and availability information presented in this section is taken from Texas Department of Water Resources documents (TDWR, 1984a and 1984b).

A total of 17.9 million acre-feet of water was used in Texas in 1980 for the following purposes: agriculture (72.5% of total), municipal (15.8%), manufacturing (8.5%), steam-electric power generation (1.9%), and mining (1.3%). Of the 17.9 million acre-feet total water used, 10.9 million acre-feet (61%) was from ground water and 7.0 million acre-feet (39%) was from surface water. The 7.0 million acre-feet of surface water, which was essentially all from reservoirs, was divided between uses as follows: agriculture (55.4%), municipal (21.7%), manufacturing (18.1%), steam-electric power generation (3.9%), and mining (0.9%). This data is for withdrawals except steam-electric power data includes only consumptive use. The actual withdrawals for cooling water are many times greater than consumptive use. A significant portion of agricultural, municipal, and manufacturing withdrawals become return flows.

The TDWR has estimated the total firm yield provided by the reservoirs in Texas to be about 11.0 million acre-feet per year. Thus, the present use of surface water state-wide is about 64 percent of the firm yield. Most of the remaining firm yield is committed for expanding municipal and industrial needs during the next 20 to 30 years.

The State of Texas is experiencing exceptionally rapid population and economic growth compared to the nation as a whole. The population of the state has grown from 3.0 million in 1930 to more than 15 million in 1983. The Texas Department of Water Resources projections shown in Figure 1 indicate that the population will increase to between 19.6 and 21.2 million in the year 2000 and between 28.3 and 34.3 million in 2030. Texas has a broad-based industrial, service, trades, energy, and agricultural economy which is growing along with the population. Dependable supplies of suitable quality water are necessary to support the economic and population growth of the state. Present and projected future requirements for each type of use are presented in Table 3. Past and projected future water use and source of supply are shown in Figure 2.

More than 50 percent of Texas is underlain by seven major aquifers and sixteen minor aquifers. Collectively, these aquifers receive an average annual natural recharge of about 5.3 million acre-feet and contain about 431 million acre-feet of water in storage that is recoverable using conventional water well technology. About 89 percent of the recoverable ground water is in the Ogallala Aquifer in the High Plains. For most of the aquifers, water withdrawal is occurring at a greater rate than recharge. Of the 10.8 million acre-feet withdrawn from ground water in 1980, 5.5 million acre feet was from storage. Ground water mining is causing water-level declines, decreased well yields, land subsidence, and saline water encroachment. By the year 2000, if current water use trends continue, the state's aquifers are projected to be capable of supplying about 6.8 million acre-feet annually, or about 63 percent of the present level. Consequently, greatly increased demands will be placed upon surface water reservoirs.

Both ground and surface water sources supply the full range of uses throughout the state. However, the bulk of the total ground water use in the state is for irrigation in the High Plains from the Ogallala Aquifer. Few major surface water reservoirs or good reservoir sites exist in this area. About half of the irrigation from surface water occurs in the lower Rio Grande

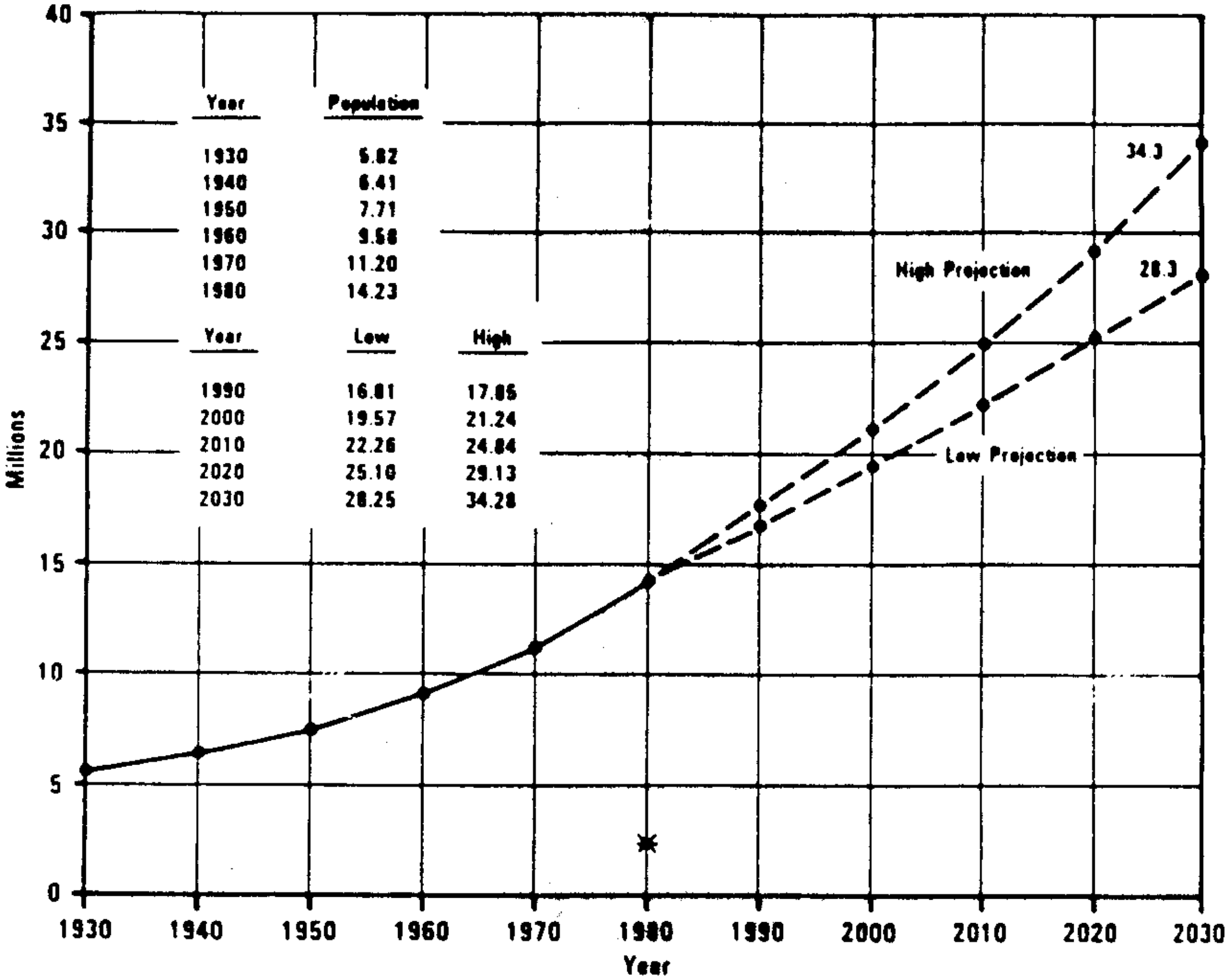


Figure 1

TDWR Population Projections

Table 3
Texas Department of Water Resources Water Use Projections

1980 Reported Water Use in Texas in acre-feet/year

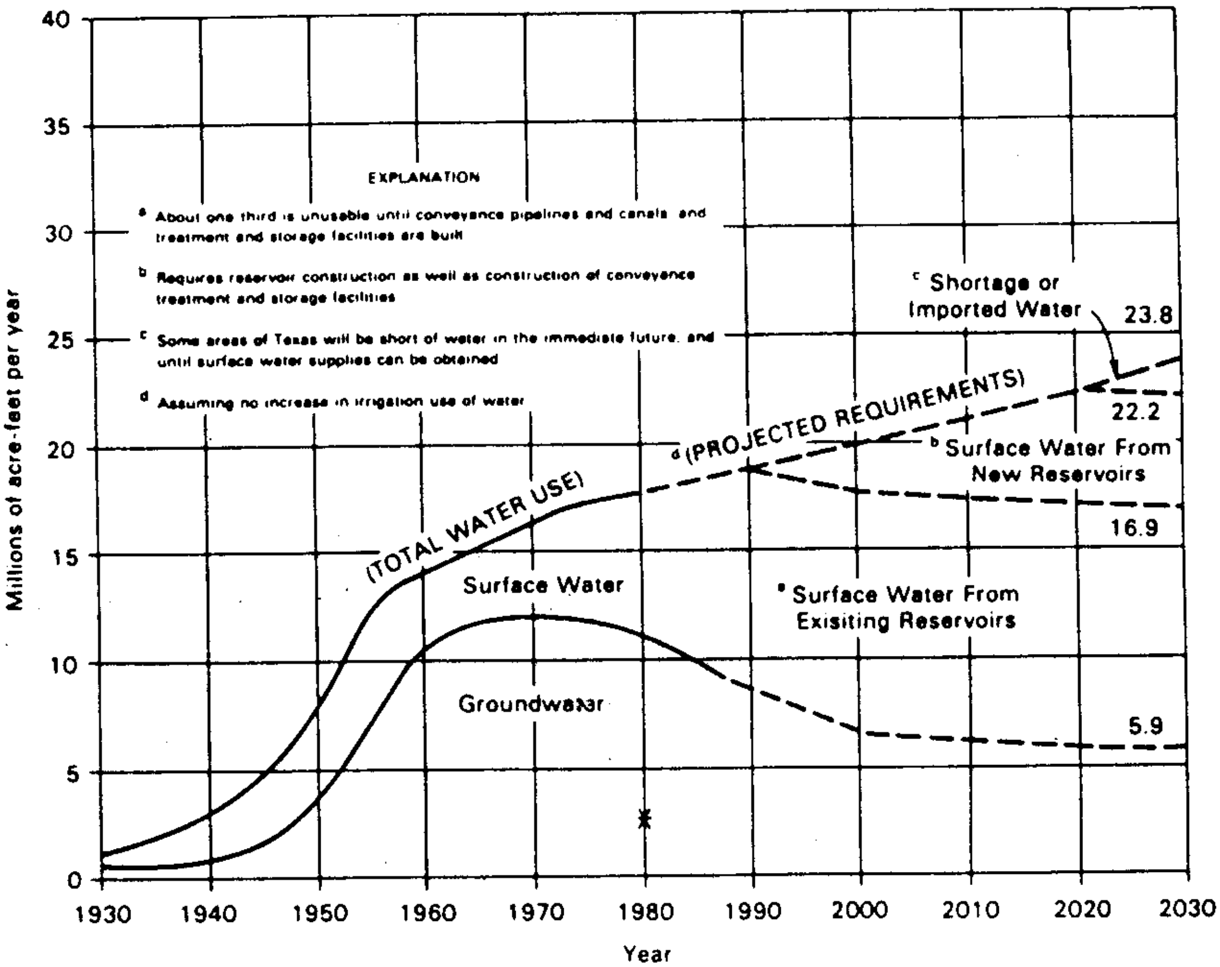
municipal and domestic	2,813,000
manufacturing	1,520,000
mining	239,000
steam-electric	330,000
agricultural	12,851,000
total	17,853,000

Projected Water Use in acre-feet/year

Year 2000	Low	High
municipal and domestic	3,512,000	5,081,000
manufacturing	2,407,000	2,718,000
mining	268,000	268,000
steam-electric	717,000	817,000
agricultural	10,427,000	16,543,000
total	17,331,000	25,425,000
Year 2030	**Low**	**High**
municipal and domestic	5,059,000	8,178,000
manufacturing	4,231,000	5,014,000
mining	387,000	387,000
steam-electric	1,119,000	1,417,000
agriculture	11,385,000	15,351,000
total	22,181,000	30,347,000

Note: In addition, estimated freshwater inflow requirements for Texas bays and estuaries range from a low (survival limit) of 4.7 million acre-feet annually to a high (enhancement) of 13.6 million acre-feet annually.

Source: Texas Department of Water Resources (1984).



Source: Texas Department of Water Resources (1984)

Figure 2
Water Use and Source of Supply Projections

Valley using water regulated by International Falcon Reservoir. Much of the remaining surface water irrigation is concentrated in the coastal areas of the eastern half of the state. Both ground water and surface water presently provide significant supplies for municipal and industrial use, but a significant shift toward a greater reliance on surface water is underway.

From 1980 to 2000, water requirements under drought conditions for non-agricultural uses are projected to increase from about 5.0 million acre-feet to 8.8 million acre-feet annually. Of the 8.8 million acre-feet, approximately 6.5 million acre-feet per year or 73 percent will be required in the 26 Standard Metropolitan Statistical Areas (SMSAs). About 62 percent of the current water use in the SMSAs are from developed surface water sources. By the year 2000, because of physical and economic problems related to overdraft or mining of ground water, about 80 percent of the 6.5 million acre-feet water requirement for the SMSAs will have to be supplied from developed surface-water resources, some of which are located in neighboring basins (TDWR, 1984b).

Conservation Purposes

All of the major reservoirs in Texas except three contain conservation storage capacity (Wurbs, 1985). The primary conservation purposes served are municipal, industrial, and agricultural (irrigation) water supply, cooling water for steam-electric plants, hydroelectric power generation, and recreation. Reservoir operation involves both complementary and conflicting or competitive interactions between these purposes. Numerous municipal, industrial, and irrigation users are dependent upon the limited resource water. Allocation between competing users is governed by the water law of the state. Hydroelectric power can often be generated with water that is released for downstream municipal, industrial and agicultural uses. In other cases, water may be released specifically and only for hydroelectric power generation. Reservoir recreation is extremely popular and a major consideration in reservoir operation in Texas. Recreation is generally complementary with the other conservation purposes. However, since operation for recreation essentially means maintaining a full pool without fluctuations in water surface level, releases and withdrawals for other purposes can be detrimental to recreation uses.

Water Supply

Municipal and/or industrial water supply is provided by 163 of the 184 conservation reservoirs. Irrigation is a designated purpose of many of the major reservoirs. Most of the reservoirs providing water for irrigation also supply municipal and industrial uses. About half of the irrigation from surface water occurs in the lower Rio Grande Valley using water regulated by International Falcon and Amistad Reservoirs.

Water supply withdrawals are made at many projects through pumping plants with intake structures located in the reservoir. In many other cases, releases are made through outlet works and spillway structures to be withdrawn from the river at downstream diversion facilities. The water may be actually withdrawn at locations several hundred river miles below the dam from which it was released. Although most of the surface water used in the state is used within the river basin from which it originates, significant interbasin transfers do occur, particularly to the coastal basins. A majority of the water supply reservoirs are operated as individual units to supply specific customers. However, a number of reservoirs are operated as systems with some degree of interaction between the component reservoirs. System operation typically involves maintaining a balance between storage depletions and water surface fluctuations in the component reservoirs. Hydroelectric power generation is also a concern in system operation. Releases are coordinated to meet water supply demands while minimizing the amount of water bypassing the turbines.

Reservoir operation procedures for water supply purposes are based essentially on meeting water demands subject to institutional constraints related to water rights, project ownership, and contractual agreements. The complex organizational framework for water supply operations involves a multitude of water users and suppliers working under various contractual arrangements. Water suppliers may either own and operate reservoirs or contract with other reservoir owners for storage capacity or water use. A number of entities both own and operate their own reservoirs and contract with others for the use of additional capacity.

Water use permits are administered by the Texas Department of Water Resources in accordance with the water law of the state. Permits may involve a variety of arrangements. Permits may be regular, seasonal or temporary, or emergency in nature. Special provisions may be made for special circumstances. The legal right to use or sell the water from a reservoir is usually granted to the owner prior to construction of the project. Many reservoirs are owned and operated by cities to provide water to their citizens for domestic, public, and commercial use. The city holds the permit or water right and sells the water to its citizen customers. Another common case is a reservoir or system of several reservoirs owned and operated by a river authority which sells the water to a number of cities, industries, and/or farmers. The river authority holds the permit or water right. The cities, industries, and farmers purchase the water from the river authority without having to obtain a water right permit through the TDWR. The river authority operates the reservoirs to meet its contractual obligations to its customers. The federal government does not get involved with water rights. The nonfederal project sponsors which contract for the conservation storage in federal projects are responsible for obtaining the appropriate water rights permits through the TDWR.

Individual farmers, industries, and cities also hold water rights permits not associated with reservoirs. In several of the river basins, a number of reservoir operators, all holding appropriate water rights permits, operate reservoirs in the same basin. Reservoir operators are often required to make releases, typically not exceeding inflows, to allow downstream users not associated with the reservoir access to the water for which they are legally entitled.

Hydroelectric Power

In 1980, total flow through the turbines of the state's 21 hydroelectric power plants exceeded 11 million acre-feet (TDWR, 1984b). Although large volumes of water are used for hydroelectric power generation, the water is not consumed and is usually used downstream for other purposes after passing through the turbines. At several of the hydroelectric plants, reservoir water diverted through the turbines is strictly limited to releases being made for other water supply or flood control purposes. At some projects, hydropower releases may be in excess of those needed for other purposes, but the multiple purposes are still closely coordinated. Several of the hydroelectric plants are located downstream of other plants such that the same water flows through two or more turbines. Hydroelectric power accounts for about 0.6 percent of the electrical power produced in Texas, with most of the electricity being produced by natural gas, and to a lesser extent coal, fired thermal electric plants. Hydroelectric power is used primarily for peak loads.

Instream Flow Maintenance

Instream flow needs include maintenance of sufficient streamflow for water quality, fish and wildlife habitat, livestock water, river recreation, and aesthetics. Water law and reservoir operation practices have traditionally favored offstream needs over instream needs. Releases for hydroelectric power and also water supply releases which are withdrawn from the river for municipal, industrial, or agricultural use at significant distances below the dam contribute to instream environmental needs as well. Operating procedures for some reservoirs include providing minimum instream flow levels for maintenance of fish and wildlife habitats. Some reservoirs have multi-level outlet works which allow selective blending of discharge waters for optimal downstream water quality. The role of reservoirs in contributing toward the maintenance of desirable levels of freshwater inflows to the state's bays and estuaries has recently received considerable attention and will likely continue to be scrutinized in the future.

Flood Control Operations

Whereas conservation operations throughout the state are the responsibility of a multitude of entities, the responsibility for flood control operations is highly centralized. The International Boundary and Water Commission is responsible for flood control operations of Falcon and Amistad Reservoirs on the Rio Grande River. These two multiple purpose projects contain 2.7 million acre-feet of flood control storage. The 12,600 acre-foot Olmos Reservoir is a flood control only project owned and operated by the City of San Antonio. It is the smallest, oldest, and only nonfederal project of the major reservoirs containing flood control storage. The U.S. Army Corps of Engineers is responsible for the 15.9 million acre-feet of flood control capacity in the remaining 32 flood control reservoirs.

The discussion here is limited to controlled storage. Releases are controlled by the operator through the use of spillway and outlet works gates. All of the large flood control reservoirs have gated spillways and/or outlet works. Numerous other small uncontrolled flood retarding and detention basin structures are in use throughout the state. The ungated outlet structures are designed with limited discharge capacities which result in outflow rates being less than inflow and storage occurring during a flood event. Streamflows are automatically reduced without requiring release decisions to be made by an operator. These small uncontrolled flood control reservoirs are not addressed in this report.

Corps of Engineers Reservoirs and Organizational Structure

The Corps of Engineers is organized with geographical divisions which are further subdivided into districts. District offices report to division offices which in turn report to the Office of the Chief of Engineers in Washington. Texas is located in the five-state Southwestern Division. The division office is in Dallas. The Galveston District is responsible for Corps of Engineers activities in the coastal area of the state. Tulsa District includes the Canadian and Red River Basins. The remainder of the state is in the geographical jurisdiction of the Fort Worth District. The Corps of Engineers operates and maintains the reservoir projects which it has constructed. The Corps of Engineers is also responsible for flood control operations at projects constructed by the Bureau of Reclamation. These projects are actually operated and maintained by local sponsors in association with the Bureau of Reclamation. However, whenever water is in the flood control pool, releases are made as directed by the Corps of Engineers.

The Corps of Engineers reservoir projects can be categorized as follows. The Galveston District owns and operates Addicks and Barker Reservoirs, which are flood control only projects with a combined capacity of 411,500 acre-feet. Texoma and Pat Mayse are multiple purpose reservoirs with 2.7 million acre-feet of flood control capacity which are owned and operated by the Tulsa District. Lake Kemp was originally constructed and continues to be owned and operated by the City of Wichita Falls. In 1974, the Tulsa District completed reconstruction of the dam to insure its safety and to provide a specific allocation of 284,300 acre-feet for flood control. The City makes releases from the flood control pool of Lake Kemp as directed by the Tulsa District. Lake Meredith was constructed by the Bureau of Reclamation and is operated by the Canadian River Municipal Water Authority. Releases from the 543,200 acre-foot flood control pool are made as directed by the Tulsa District. Travis and Twin Buttes likewise were constructed by the Bureau of Reclamation and are operated by the Lower Colorado River Authority and City of San Angelo, respectively. The Fort Worth District is responsible for releases from the flood control pools which have a combined capacity of 1.2 million acre-feet. The Fort Worth District owns and operates the following 21 multiple purpose reservoirs which contain a total of 10.2 million acre-feet of flood control capacity: Wright Patman, Whitney, Rayburn, Belton, Waco, Lewisville, Stillhouse Hollow, Canyon, Somerville, Lake O' the Pines, Proctor, Fischer, Lavon, Grapevine, Benbrook, Granger, Navarro Mills, Georgetown, Aquilla, Bardwell, and Hords Creek. Lake O' the Pines and Wright Patman Reservoirs were operated by the New Orleans District until 1979 when jurisdiction for the Texas portions of the Cypress Creek and Sulphur River Basins were transferred to the Fort Worth District. Ray Roberts, Joe Pool, and Cooper Reservoirs are multipurpose projects with 519,300 acre-feet of flood control capacity which are presently under construction by the Fort Worth District (Wurbs, 1985).

The Fort Worth District is responsible for about 58 percent of the flood control storage capacity of the major reservoirs in the state. Fort Worth, Tulsa, and Galveston Districts operate a total of about 86 percent of the flood control storage capacity.

A reservoir control center in the Southwestern Division office in Dallas provides overall management and coordination of reservoir operation activities in the several districts of the division. The district offices are responsible for the actual operation of the reservoirs. Each district organization includes an operations division responsible for operation and maintenance of completed projects. However, real-time reservoir regulation and associated water control activities are the responsibility of a reservoir control unit which is a part of the hydraulics and hydrology branch of the engineering division. Thus, a central reservoir control organization within the district office is responsible for determining the releases to be made at all of the reservoirs within the district. Reservoir managers and supporting personnel at the individual reservoir projects operate the spillway and outlet works gates as instructed by the district office. Telecommunications between the reservoir control unit and the reservoir project offices occur at least daily and can be essentially continuous during major flood events. Emergency operating procedures are established for each project as a contingency in case communications should be disrupted during a flood.

The projects have two general types of outlet structure configurations. A number of the projects have an uncontrolled broadcrested or ogee spillway, with the crest elevation at the top of flood control pool, combined with an outlet works structure consisting of a gated intake structure, conduit through the dam, and downstream stilling basin. The gates are located at various depths below the top of conservation pool. Other projects have a controlled spillway with a set of several tainter gates. Tainter gates (also called radial gates) rest upon the spillway crest when fully closed. A gate is opened by lifting, with the water flowing under the gate and over the spillway crest. Controlled releases from the flood control pool are made by raising the tainter gates. Sluices with gates at lower elevations are also provided for relatively small releases.

Flood control and conservation pools in a multiple purpose reservoir are designated by set pool elevations. The top of conservation (bottom of flood control) pool, top of flood control pool, and maximum design water surface are key pool levels or elevations in flood regulation schedules. Releases are made from the conservation pool for water supply purposes at the request of the local sponsors which have contracted for the storage. The flood control pool is the space between the top of conservation pool and the top of flood control pool. Releases from the flood control pool are regulated by opening and closing spillway and/or outlet works gates. Surcharge storage occurs whenever the flood control pool is full and inflows exceed discharges through the spillway. The maximum design water surface is the critical condition for which the dam and appurtenant structures were designed. Consequently, release policies are predicated on never under any circumstances allowing surcharge storage to exceed the design water surface.

Reservoirs designed and constructed by the Corps of Engineers are normally sized to contain a flood with an associated recurrence interval in the range of 50 to 100 years, or in some cases greater, without exceeding the capacity of the flood control pool. Consequently, filling to the top of flood control pool is an infrequent event. Many of the projects have never had the flood control pool completely full. This is not necessarily the case for multipurpose projects constructed by others for which the Corps of Engineers is responsible for flood control operations.

Corps of Engineers Operating Procedures

Each project has operating procedures which are documented in a reservoir regulation manual. A regulation schedule specifies the releases to be made under various conditions. Formulation or modification of a plan of operation requires extensive hydrologic, hydraulic, economic, and environmental studies. The plan of operation is established during project planning and design. Modifications in the operating procedures for operational projects are made as required to reflect experience gained in actual operation or changed conditions such as construction of additional projects in the basin. However, operation procedures tend to remain fairly constant over time.

Flood control regulation schedules are developed to address the particular conditions associated with each individual reservoir and river basin. Peculiarities and exceptions to standard operating procedures occur at various projects. However, the regulation schedules for all the projects were developed following essentially the same guidelines, as outlined in the Corps of Engineers manual on reservoir regulation (USACOE, 1959) and have the same general strategy. An overview of flood control operating procedures is provided below.

The flood control regulation schedule for a reservoir actually consists of two schedules. Both schedules are followed, and the one requiring the largest release rate controls for a given set of conditions. The regular schedule, which usually controls, is based on the assumption that ample storage capacity is available to handle the flood without special precautions being necessary to prevent the water surface from rising above the top of flood control pool. Operation is switched over to an alternative schedule during extreme flooding conditions when the anticipated runoff from a storm is predicted to exceed the controlled capacity remaining in the reservoir. If the water surface level significantly exceeds the top of flood control pool, downstream damages will necessarily occur. The objective is to assure that reservoir releases do not contribute to downstream damages as long as the storage capacity is not exceeded. However, for extreme flood events which would exceed the reservoir storage capacity, moderately high damaging discharge rates beginning before the flood control pool is full are considered preferable to waiting until a full reservoir necessitates much higher release rates.

The regulation schedule for Waco Reservoir is reproduced in Figure 3 to illustrate the general procedure (USACOE, 1971). This type of schedule has been developed for each flood control reservoir to guide real-time flood operations whenever the storage capacity is predicted to be exceeded. In this case, release decisions are based on a current reservoir water surface elevation and inflow. The required outflow for a given reservoir elevation and inflow is read from the graph. If this outflow is less than that specified by the regular schedule, the regular schedule is followed. Otherwise, the gates are operated to release the outflow indicated by the graph.

The family of curves presented in the figure represent a compromise between two conflicting considerations in regard to handling extreme flood events which would significantly exceed the flood control storage capacity of the reservoir. The regular schedule discussed below is based on not making releases unless downstream streamflows are below damaging levels. This regular schedule conceivably could be followed until the flood control pool fills. However, after the flood control pool is full, tremendously high

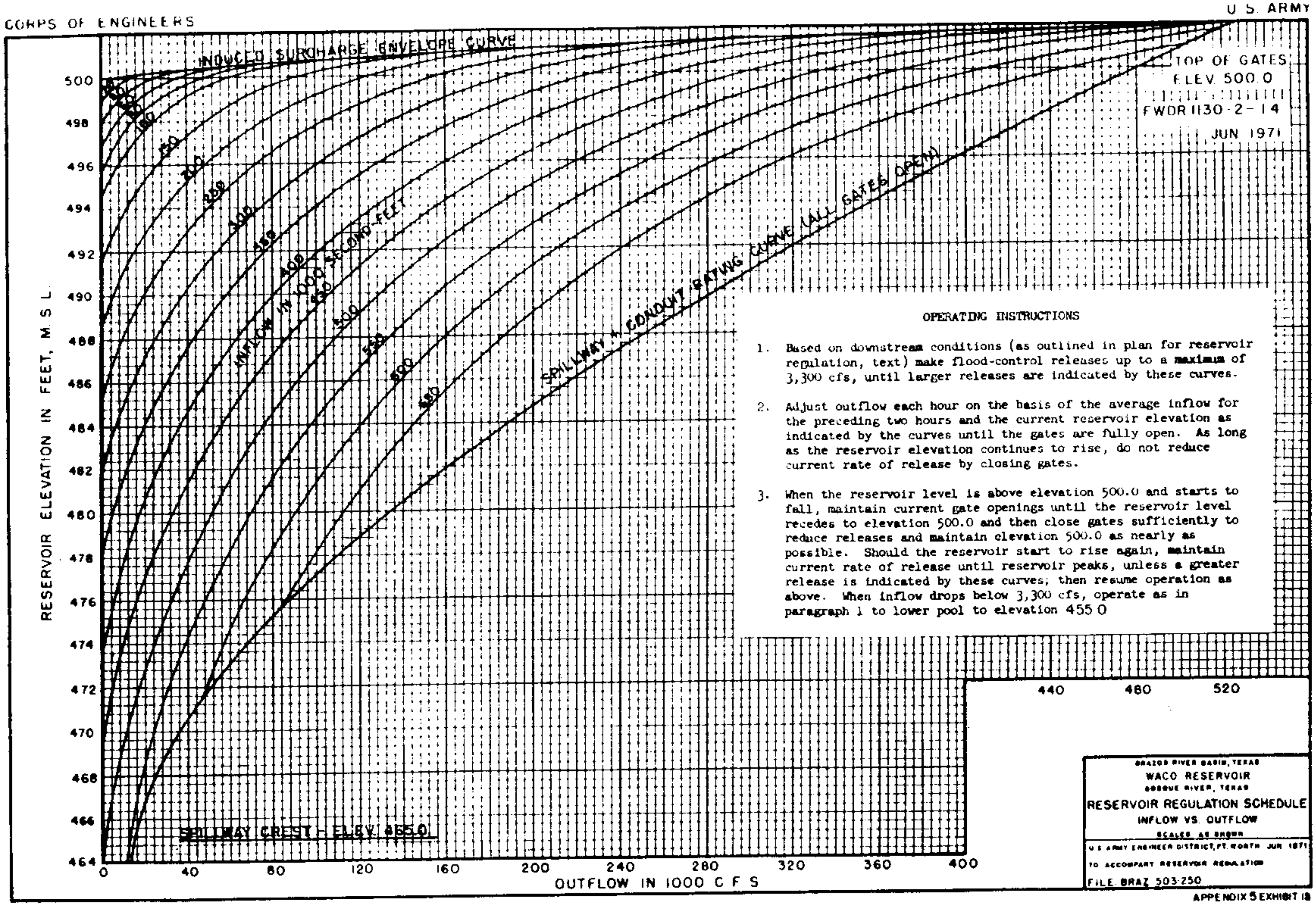


Figure 3 - Reservoir Operation Schedule

discharge rates may be required to prevent the surcharge storage from exceeding the design water surface. The much higher peak release rate necessitated by this hypothetical operation policy can be expected to be much more damaging than a lower release rate with a longer duration beginning before the flood control pool is full. On the other hand, an operator would not want to make damaging releases early in a storm and later find that a portion of the flood control pool remained empty during the storm. Although streamflows that will occur several hours or days in the future are typically forecast during real-time operations, future flows are still highly uncertain.

The regulation schedule curves are developed based on estimating the minimum volume of inflow that can be expected in a flood, given the current inflow rate and reservoir elevation. Having estimated the minimum inflow volume to be expected during the remainder of the flood, the outflow required to limit storage to the available capacity is determined by mass balance computations. For a given current inflow rate, the minimum inflow volume for the remainder of the storm is obtained by assuming the inflow hydrograph has just crested and computing the volume under the recession side of the hydrograph. For conservatively low inflow volume estimates, the assumed recessive curve is made somewhat steeper than the average observed recession. The complete regulation schedule which allows the outflow to be adjusted on the basis of the current inflow and empty storage space remaining in the reservoir is developed by making a series of computations with various assumed values of inflows and amounts of remaining storage available.

As previously indicated, the flood control regulation schedule for a reservoir actually consists of two schedules. The regular schedule is followed as long as the indicated releases are greater than the outflow values read from the graph discussed above. The regular schedule is based on downstream flooding conditions. Nondamaging flow rates and stages are specified at selected index locations, called control points, which are representative of the damage potential in the associated reach of channel and flood plain. Nondamaging flow rates are equal to or closely related to bankful stream capacities. U.S. Geological Survey stream gaging stations are located at the control points. Releases are made to empty the flood control pool as quickly as possible without exceeding the allowable flow rates at each downstream control point. The regulation schedule consists of specified flow rates to be maintained at the designated control points.

When a flood occurs, the spillway and outlet works gates are closed. The gates remain closed until a determination is made that the flood has crested and flows are below the nondamaging levels specified for each of the control points. The gates are then operated to empty the flood control pool as quickly as possible without exceeding the allowable flows at the control points. Normally, no flood control releases are made if the reservoir level is at or below the top of conservation pool. However, if flood forecasts indicate that the inflow volume will exceed the available conservation storage, flood control releases from the conservation storage may be made if downstream conditions permit. The idea is to release some water before the stream rises downstream, if practical, for a forecasted flood.

For many reservoirs, the allowable flow rate associated with a given control point is constant regardless of the reservoir surface elevation, assuming the outflow still exceeds the value specified by the previously discussed graph illustrated by Figure 3. At other projects, the flood control pool is subdivided into two or three zones with the allowable flow rates at one or more of the control points varying depending upon the level of the reservoir surface with respect to the discrete alternative zones. This allows stringently low flow levels to be maintained at certain locations as long as only a relatively small portion of the flood control pool is occupied, with the flows increased to a higher level, at which minor damages could occur, as the reservoir fills. The variation in allowable flow rates at a control point may also be related to whether the reservoir level is rising or falling.

A reservoir is operated based on maintaining flow rates at several control points located various distances below the dam. The most downstream control points for a number of projects are well over a hundred miles below the dam. Lateral inflows from uncontrolled watershed areas below the dam increase with distance downstream. Thus, the impact of the reservoir on flood flows decreases with distance downstream. Operating to downstream control points requires streamflow forecasts. Flood attenuation and travel time from the dam to the control point and inflows from watershed areas below the dam must be estimated as an integral part of the reservoir operating procedure.

Most of the flood control reservoirs are components of basinwide multireservoir systems. Two or more reservoirs located in the same river basin will have common control points. A reservoir may have one or more control points which are influenced only by that reservoir and several other control points which are influenced by other reservoirs as well. Reservoirs in a system are operated, to the extent practical, to maintain approximately the same percentage of flood-control storage utilized in each reservoir. Releases from all reservoirs, as well as runoff from uncontrolled watershed areas, must be considered in forecasting flows at control points.

Maximum allowable rate of change of reservoir release rates are also usually specified. Abrupt gate openings causing a flood wave with rapid changes in stage are undesirable.

The top of conservation and flood control pool elevations are established during project planning and design and normally remain constant thereafter. A multipurpose reservoir operating policy in which conservation and flood control operations are differentiated by a designated top of conservation (bottom of flood control) pool elevation is sometimes referred to as a rule curve. The rule curve essentially specifies the top of conservation pool elevation, which in general may vary monthly or seasonally. However, all the multiple purpose flood control reservoirs operated by the Corps of Engineers in Texas, except two, have constant top of conservation pool elevations with no seasonal variation. Lake O' the Pines and Wright Patman Lake, in the northeast corner of the state, are the two projects with seasonal rule curves. The operating rule curve for Lake O' the Pines provides for raising the top of conservation pool 1.5 feet from mid-May through mid-September for recreation purposes (USACE, 1974). The operating rule curve for Wright Patman varies significantly during the year in response to an interim operating agreement with the conservation storage sponsor to provide additional municipal and industrial water supply (USACE, 1974). The top of conservation pool is constant from November through March and varies with date from April through October. The top of conservation pool peaks on June 1 at a level 6.9 feet above the winter pool level.

Other Projects

The International Boundary and Water Commission operates Falcon and Amistad reservoirs with designated flood control pools similar to the Corps of Engineers. Top of conservation and flood control pool elevations are set and normally used in operating the reservoirs. The top of conservation pool elevations can be, at the discretion of the Commission, temporarily raised for seasonal rule curve type operation. However, the optional encroachment into the flood control pool does not necessarily occur routinely each year and the magnitude of encroachment can be varied within a fixed maximum limit. Flood control operations are based on storing as much flood water as possible without endangering the integrity of the dam.

Olmos Reservoir, located on Olmos Creek, is owned and operated by the City of San Antonio. Impounded flood waters are stored until releases can be made without damaging downstream property. During nonflood periods when the reservoir is empty, the pool area serves as a park and playground.

Problems, Issues, and Directions

Determining the amount of water to store and to release or withdraw from a reservoir or system of several reservoirs is a complex decision making process involving numerous physical, hydrologic, environmental, economic, legal, institutional, political, and public relations considerations. Each reservoir project or system has its particular operating complexities and difficulties. Several major issues and problem areas pertinent to reservoir operation in general in the state are cited in this subsection. Present and potential future directions in reservoir operations are also discussed.

Multiple Uses and Users

Many of the complexities associated with surface water management involve allocation of limited reservoir storage capacity and water releases to competing purposes and users. Minimizing conflict and dealing with controversy is an integral part of reservoir management. Project purposes may be either complementary or conflicting depending on the circumstances.

As previously discussed, an unmanageable water rights system has been a major complicating factor in managing the surface water resources of the state. The new permit system should greatly enhance surface water management. Many conflicts over water rights have been resolved. The quantities of water for which various entities hold permits are known. Estimates of the amount of water still available for future water rights appropriations are also known. A major problem at the present time is assuring that water is actually used as allocated on paper. The Rio Grande is presently the only river basin with a water master system. Other basins will likely have water masters in the future. Without a strict enforcement and accounting system, the water may not necessarily be actually used in accordance with the adjudicated water rights or permits issued.

Minimizing adverse impacts on recreation while fulfilling other project purposes is typically a significant consideration in multiple purpose reservoir operation in the state.

Increasing hydroelectric power generation at existing reservoir projects has received considerable attention during the past decade, both nationwide and in Texas. Investigations continue regarding potentialities for adding more generating units and increasing production from existing units in the state. Availability of water is the primary limiting factor in hydroelectric power production. Additional savings in natural gas and lignite burned in steam-electric plants could be achieved if additional water was made available for hydroelectric power.

Water law and reservoir operation practices have traditionally favored offstream needs over instream needs. Releases for hydroelectric power and water supply releases which are withdrawn from the river for municipal, industrial, and agricultural use at significant distances below the dam contribute to instream environmental needs as well. Operating procedures for some reservoirs include providing minimum instream flow levels for maintenance of fish and wildlife habitats. Instream needs are now receiving increasingly more attention as streamflows are depleted through storage and withdrawal.

Maintenance of freshwater inflows to bays and estuaries is currently a major issue in Texas. Seven major and several minor estuaries are located along the 400 miles of Texas coastline. The inflow of freshwater is widely recognized as an essential factor in maintaining the biological productivity of the bays and estuaries. The TDWR and others have studied the importance of freshwater to each estuarine system in the state and developed estimates of quantities and seasonal timing of freshwater inflows needed (TDWR, 1984). The role of reservoirs in contributing toward the maintenance of desirable levels of freshwater inflows to the state's bays and estuaries has recently received considerable attention and will likely continue to be scrutinized in the future.

Traditional Problems

Evaporation and sedimentation are natural processes which adversely affect reservoir operation. Nature collects a tax on stored water in the form of evaporation. Over four million acre-feet of water per year is evaporated from the major reservoirs of Texas. This is a very significant water loss when compared to the seven million acre-feet per year of surface water used for beneficial municipal, industrial, and agricultural purposes.

Rivers in Texas transport large volumes of sediment produced by erosion in the contributing watershed, particularly during major rainfall and flood events. Streambank erosion and aggradation also occurs. Reservoirs in Texas are efficient sediment traps. Accumulation of sediment deposits significantly reduces reservoir storage capacity over time. Reservoir design typically includes providing additional capacity to accommodate 100 years of sedimentation.

Hydrolectric power operations typically involve frequent relatively rapid fluctuations in reservoir pool levels and release rates. Erosion of the reservoir shoreline and downstream channel banks sometimes accompany hydropower releases. Complaints are not uncommon from downstream riparian property owners concerned about losing their land to streambank erosion. Due to the reservoir acting as a sediment trap, reservoir releases have a lower suspended sediment load and correspondingly greater erosion capacity than unregulated streamflow. This adds to the problem of hydropower releases causing downstream erosion.

Downstream channel encroachments and other limitations on channel capacities are a major problem for flood control operations. Problems may be caused by an incorrect initial estimate of nondamaging discharge levels, decrease in channel capacity due to natural erosion and aggradation processes, or encroachments as people locate activities near or in the river. In some situations, someone will be adversely affected to some extent by almost any discharge level.

Flood control operations are based on reducing peak flows which means that reservoirs result in long flow durations at lower flow rates. In some cases, people would prefer an uncontrolled high magnitude but short duration flood. For example, a low water crossing for a road might be inundated much longer with than without a flood control reservoir. People are concerned about how many days the road is closed rather than the peak depth of inundation. Streambank caving is also sometimes attributed to maintaining bankful flows for long periods of time. Extended duration flows can also delay drainage of water ponded behind levees.

Changing Conditions

Rapid population and economic growth and depleting groundwater reserves are resulting in continually increasing demands being placed upon the surface water resources of the state. However, economic, financial, political, and environmental feasibility of constructing new reservoirs is much more difficult to achieve now than in the past (Wurbs, 1985). The past construction era of water resources development has transitioned into the present management era. Consequently, an increased focus on optimizing the operation of existing projects to meet increasing demands can be expected.

Reservoir storage capacities and operating policies are generally established prior to construction and tend to remain constant thereafter. However, public needs and objectives and numerous factors affecting reservoir effectiveness significantly change over the years. The increasing pressure to use existing limited storage capacity as efficiently and beneficially as possible should encourage periodic reevaluations followed, whenever warranted, by changes in reservoir operating policies.

As illustrated graphically in Figure 2, water supply needs are increasing with population and economic growth, and depleting groundwater supplies are resulting in an increasing reliance on surface water. Other factors affecting the effectiveness of reservoir operating policies change over time as well. The economics of alternative sources of electrical energy has focused attention during the past decade on increasing hydroelectric power generation at existing reservoirs through improved operating procedures and/or additional generating units. Watershed and flood plain conditions change over time. Construction of numerous small flood retarding dams by the Soil Conservation Service and other entities in the watersheds of major reservoirs have reduced flood inflows to the reservoirs. Construction of numerous small ponds for recreation or watering livestock have also decreased reservoir inflows and yields. Increased runoff caused by watershed urbanization is significantly contributing to flooding problems in certain locations. The existing flood control reservoirs were planned and designed based on the expectation of ever increasing intensification of flood plain land use. However, the National Flood Insurance Program has resulted in zoning and regulation of 100-year flood plains. With stringent flood plain management, susceptibility to flooding could actually decrease over time as existing activities choose to leave the flood plain and regulation prevents other activities from moving into the flood plain. Reservoir sedimentation reduces available storage capacity. Construction of additional reservoirs, as well as other related types of projects such as conveyance facilities, flood control levees and channel improvements, and electric power plants, affect the operation of existing reservoirs. Technological advancements in hydrologic data collection, streamflow forecasting, system modeling and analysis, and computer technology provide opportunities for refining operating policies.

Expanded Management Strategies and Analysis Capabilities

Under present and projected future conditions of economic development and water use, expanded management strategies and analysis capabilities are required to effectively prepare for the next severe drought while, at the same time, optimizing the present beneficial use of limited storage capacity for the various purposes. The present study focused upon reallocation of storage capacity between flood control and water supply purposes. Storage reallocation is discussed in the following chapter. Several other general approaches which potentially could be adopted, or emphasized more in the future, to increase the effectiveness of the existing reservoirs are cited below.

Automated hydrometeorological data collection and management systems represent a major area of technology advancement in real-time reservoir operation for flood control. The Fort Worth District of the Corps of Engineers has recently installed data collection platforms at each of the gaging stations used in reservoir operations. Streamflow and rainfall readings are automatically communicated from the platforms via satellite to a computer system assessed by the reservoir control unit. The Lower Colorado River Authority recently installed a similar system to monitor their river basin activities. Advanced hydrometeorological data collection and streamflow forecasting systems could be particularly useful for operating multipurpose reservoirs to reduce flood damages to the maximum extent possible while devoting a larger portion of the storage capacity to conservation purposes or for operating conservation only reservoirs to reduce flood damages.

Reservoir operation involves risks and consequences associated with failing to meet various levels of water supply demands and failing to provide various levels of flood protection. Planning, design, and operation of flood control reservoirs have traditionally been based on analyzing the risks (probabilities) and consequences (damages) associated with alternative plans of action. However, planning, design, and operation of water supply reservoirs have relied primarily on the concept of firm yield to quantify risk. Firm yield does not provide a very meaningful measure of the likelihood of failing to meet various demand levels or the consequences. A major emphasis of the present study was the development of a procedure for quantifying the risks and economic consequences of failing to meet water demands. Although the procedure was formulated in the context of evaluating storage reallocation plans, the concepts are pertinent to water supply planning and management in general.

A major current water policy emphasis is the need to shift to a greater reliance on demand management and more efficient water use. Demand management could be combined with reservoir operation as integral components of a comprehensive water management process. Certain demand management strategies can be implemented essentially independently of supply management. However, emergency demand management strategies might be most effective if implemented only during times of water supply shortage. Severe long-term cutbacks in water use may not be warranted if water is flowing over the spillway of a full reservoir most of the time. However, a contingency plan is needed for when the infrequent severe drought does occur. Demand levels need to be reduced as reservoir storage is depleted. A comprehensive water management strategy combining reservoir operation with drought contingency measures could help alleviate the adverse consequences of a reservoir failing to meet certain demand levels. A certain degree of calculated risk could then be incorporated into reservoir operating policies. Demand management is an integral part of the evaluation procedure developed by the present investigation.

Conjunctive management of surface and groundwater has long been recognized as a potential strategy for increasing the beneficial use of limited water resources. Groundwater aquifers are being mined in Texas while reservoirs are full of evaporating water. Although much of the groundwater mining occurs in areas in which surface water is not available, considerable potential exists for greater coordination between ground and surface water use as the state shifts to a greater reliance on surface water. Institutional constraints including water law considerations presently severely limit conjunctive management. If and when the state moves toward greater institutional control of groundwater, conjunctive management of ground and surface water sources can be expected to become a major consideration in reservoir operation.

In the future, as water supply operation becomes more complicated, secondary yield and zoning of conservation pools may become important. Water supply releases would then be based upon reservoir levels as well as demands. If water is in the upper conservation pool zone, releases would be unrestricted. As the reservoir levels fall below certain levels, releases would be curtailed and alternative plans of action, such as demand management and increased groundwater pumpage, implemented. Thus, the reservoir operating policy would consist of conservation zones and allowable withdrawals associated with each zone.

Comprehensive system management of existing facilities and resources also has potential of reducing the risk of failing to meet water supply needs. The reliable yield which results from the coordinated joint operation of a system of several reservoirs is greater than the sum of the yields of each reservoir operated independently.

The Corps of Engineers is also interested in improved system operation for flood control. A new organization was recently established in the Tulsa District to provide hydrologic and economic computer modeling services to all the districts in the Southwestern Division. A major focus will be on developing basinwide system models for each of the major river basins in the Southwestern Division including those in Texas.

Wurbs, Tibbets, Roy, and Cabezas (1985) point out the large gap between research that has been accomplished in developing methods for analyzing reservoir reliability and optimizing release policies and the practices followed in the actual planning, design, and operation of reservoir projects. As the risk of failing to meet demands increase and reservoir operation decisions become more difficult, the potential usefulness of modeling techniques from the disciplines of water resources systems analysis and stochastic hydrology increases. Consequently, systems analysis and hydrologic modeling techniques should play an even greater role in reservoir operation in the future.

CHAPTER 3
STORAGE REALLOCATION

Reallocation of storage capacity between flood control and conservation purposes could be physically implemented simply by lowering or raising the designated top of conservation pool elevation. Storage reallocations could be between pools in a single reservoir or between reservoirs in a multiple reservoir system. Reallocations could be either long-term or seasonal. The division between flood control and conservation storage in a reservoir could be dependent upon the amount of water currently available in other reservoirs in the system or could even consider current soil moisture conditions. Another related strategy for modifying reservoir release policies is to alter the allowable flood control release rate without changing the designated storage capacity. For example, flood control releases might be limited to rates which can be beneficially used for hydroelectric power generation. Reallocations between flood control and conservation storage capacity could be warranted in two common situations in Texas: (1) conversion of a portion of the capacity in a conservation only reservoir to flood control, and (2) conversion of a portion of the flood control capacity in a multipurpose reservoir to conservation.

Flood control in Texas, as well as elsewhere in the nation, has generally been viewed as a federal responsibility. Practically all the flood control storage capacity in the state is owned and operated by federal agencies. Difficulties in financing flood control have been a major reason that reservoirs constructed by state and local entities have not included storage capacity designated for flood control. However, national water policy currently emphasizes shifting responsibilities from the federal government to the states. Consequently, the state could assume a greater role in flood control in the future which could stimulate interest in operating nonfederal reservoirs for flood control. Although new institutional arrangements might be necessary, the numerous conservation only projects in the state could conceivably also be operated for flood control. Corps of Engineers reservoirs are usually designed to contain at least 50 to 100-year recurrence interval floods without overtopping the flood control pool. Providing this degree of protection by reallocating a portion of the storage capacity in a conservation reservoir to flood control would normally not be practical due to the large storage volume required. However, lesser degrees of protection could possibly be provided while still maintaining significant conservation storage capacity.

Reallocation of a portion of the flood control storage in a multiple purpose reservoir to conservation might also be warranted under suitable conditions. Several such reallocations have occurred in conjunction with construction of new projects. Upon completion of construction of Ray Roberts Reservoir scheduled for 1986, a portion of the flood control capacity in Lewisville Reservoir will be reallocated to water supply. Flood control capacity in the new Ray Roberts Reservoir will compensate for the reallocation in the existing Lewisville Reservoir. Likewise, a reallocation of flood control to conservation capacity is planned for Wright Patman Reservoir upon completion of construction of Cooper Reservoir which is scheduled for 1991. In the interim awaiting completion of Cooper, a seasonal rule curve was implemented for Wright Patman in 1968 in which the designated top of conservation pool is raised during certain months of the year to provide additional water supply storage.

Increasing needs for conservation purposes could also justify storage reallocations in cases where new reservoirs are not compensating for the reduction in flood control capacity. For Corps of Engineers projects, the Office of the Chief of Engineers in Washington has the discretionary authority to approve storage allocation changes involving not greater than 15 percent of total storage capacity allocated to all authorized federal purposes or 50,000 acre-feet, whichever is less (USACOE, 1981). Greater changes would require authorization by the U.S. Congress. The local project sponsor would have to concur with and in most cases actually initiate the request for any modifications in operating policies. The Corps of Engineers would be responsive to modifying operating policies, if the changes could be clearly demonstrated to be in the public interest, and local interests strongly supported the changes.

In the past, public agencies and water users have not seriously pursued storage reallocation at existing projects as an alternative to developing new projects. At least, if reallocation was considered, it was determined infeasible without detailed study. However, conversion of storage capacity from flood control to hydroelectric power has received some attention in recent years. The National Hydropower Study included reallocation of storage capacity from flood control to hydropower as one of several means for increasing electric energy production (Davis and Buckley, 1984). The Cowanesque Lake

Reformulation Study completed by the Corps of Engineers Baltimore District in 1982 recommended conversion of 31 percent of the flood control storage in a major existing multipurpose reservoir to conservation purposes of hydroelectric power and municipal and industrial water supply. In Texas, the top of conservation pool of Sam Rayburn Reservoir was raised 0.4 feet in 1968 to provide additional water supply. A negligible loss of flood control benefits was associated with this very small reduction in flood control storage capacity. An additional even smaller reallocation from flood control to water supply is presently being considered for Sam Rayburn Reservoir. Denison Dam, which impounds Texoma Reservoir, was constructed in the early 1940's for flood control and hydroelectric power. A small portion of the storage capacity was reallocated to water supply in the 1950's. Additional similar reallocations of Texoma storage capacity have been considered in recent years. As discussed in the next chapter, conversion of a portion of the flood control storage in Waco Lake to conservation for municipal water supply has recently been proposed.

As discussed in the previous section on flood control operations in Texas, Lake O' the Pines and Wright Patman Lake have seasonal rule curves. The top of conservation pool elevations in International Falcon and Amistad Reservoirs can also be temporarily raised for seasonal rule curve operation. However, the optional seasonal rule curve operations for Falcon and Amistad are not necessarily implemented every year.

At this time, the examples of actual and proposed reallocations cited above are somewhat unique. However, municipal, industrial, and agricultural water users and electrical utilities will likely request storage reallocations for other reservoirs in Texas and the nation as increasing demands continue to be placed upon limited water resources.

CHAPTER 4
MODELLING RESERVOIR OPERATIONS

A major thrust of the research was the review of available modelling capabilities and adoption of selected models to the specific problem of evaluating plans for reallocating storage capacity between flood control and conservation purposes. A state-of-the-art review of systems analysis techniques applied to reservoir operation in general is documented by Wurbs, Tibbets, Cabezas, and Roy (1985). The intent of the present chapter is to provide (1) an overview summary of the types of models used in analyzing reservoir operations and (2) an introduction to specific techniques incorporated into the analysis strategy developed by the study and outlined in subsequent chapters.

Numerous mathematical models have been reported in the literature for sizing storage capacities and establishing release policies during project planning and for supporting release decisions during real-time operations. Each particular model was developed specifically for either planning or real-time applications or may be applicable in either situation. However, the present investigation addressed the somewhat different situation of evaluating plans for reallocating storage capacity in existing reservoir systems. Little attention has been directed in the literature toward reevaluating existing operating policies in response to changing public needs and conditions. A comprehensive literature review revealed essentially no models developed specifically for evaluating reallocations between flood control and conservation or otherwise considering tradeoffs and interactions between flood control and conservation purposes. However, generalized models and modelling concepts can be applied meaningfully to the analysis of storage reallocation plans even if they were not developed specifically for that particular application. The research reported herein addressed modelling of reservoir operations in general but from the perspective of identifying those modelling concepts and techniques which are pertinent to the storage reallocation problem.

Types of Models

The various types of mathematical models used in analyzing reservoir operations can be categorized as (1) simulation, (2) optimization, and (3) streamflow synthesis. A broad range of types of analyses routinely applied in the planning, design, and operation of reservoir projects are included in the category of simulation modelling. The role of optimization models is to provide the capability to search through a large number of possible combinations of values for a set of decision variables to find the decision policy which

Figure 4

Types of Simulation Models Typically Used in Analyzing Reservoir Operations

1. Hydrologic (Water Quantity)
   - rainfall-runoff (watershed models)
   - streamflow (flood routing)
   - reservoir yield and reliability
   - system operation for flood control
   - system operation for conservation purposes

2. Economic
   - flood damages
   - benefits for conservation purposes

3. Water Quality

4. Sediment Transport

maximizes or minimizes a defined objective function. Streamflow synthesis methods are used to extend and supplement historical records for developing required input data for simulation and optimization models.

A simulation model is a representation of a system used to predict the behavior of the system under a given set of conditions. Simulation is the process of experimenting with a simulation model to analyze the performance of a system under varying conditions. Although simulation only serves to analyze system performance under a given set of conditions, trial-and-error runs of a simulation model can be used to search for an optimal decision policy. However, numerous simulations may be required to achieve acceptable results, and the optimum decision may never be found. Consequently, application of mathematical programming or optimization techniques, which automatically find the optimum decision policy, to reservoir operation has received much attention.

Simulation models have been proven through practical application to be a valuable aid in sizing reservoirs and establishing operating policies. During the past twenty years, a major thrust of research and the resulting literature related to reservoir operation has been to supplement simulation models with optimization techniques such as linear programming, dynamic programming, and various nonlinear programming algorithms. The academic research community in particular, and many practitioners as well, have been very enthusiastic about applying optimization techniques to reservoir operation problems. Research in this area has dominated the water resources planning and management literature. Research results, case studies, and experience in application of optimization models in actual planning and real-time operation decisions indicate a high potential for improving reservoir operations through their use. However, optimization techniques have not yet been accepted by the reservoir planning and management community for routine use. Optimization models have played a relatively minor role compared to simulation models in regard to influencing decisions made in the planning and operation of actual projects. Simulation is the "work-horse" of reservoir system analysis. Optimization techniques provide valuable supplemental analysis capabilities for a select number of specific types of problems.

Optimum sizing of storage capacities, establishing release policies, and real-time operations are complex tasks involving numerous hydrologic, economic, environmental, institutional, and political considerations. Defining system objectives, developing criteria for quantitatively measuring system performance in fulfilling the objectives, and handling interactions and conflicts between objectives is a major area of complexity. Mathematical optimization techniques require that the real system be represented in the proper mathematical format. Representing complex project objectives and performance criteria in the required format, without unrealistic simplifications, is a particularly difficult aspect of the modeling process which limits the application of optimization techniques.

Since simulation models are limited to predicting the system performance for a given decision policy, optimization models have a distinct advantage in this regard. However, simulation models have certain advantages over optimization models from a practical applications perspective. Simulation models generally permit more detailed and realistic representation of the complex hydrologic and economic characteristics of a reservoir system. Stochastic analysis methods can be combined with simulation models easier than with optimization models. The concepts inherent in simulation tend to be easier to understand and communicate than optimization modeling concepts.

Combined use of simulation and optimization models is an effective analysis strategy for certain reservoir operation problems. Preliminary screening with an optimization model may be used to develop a manageable range of alternative decision policies for further detailed analysis with a simulation model. Another approach is for an optimization model to be embedded as a component of a complex simulation model. Likewise, an optimization model may search for an optimum decision policy while activating a simulation model to compute the objective function value for any given set of decision variable values.

Although the potential for applying optimization techniques in analyzing storage reallocation plans was investigated, the evaluation strategy developed in the present study is based strictly on simulation. The reallocation decision problem is basically to determine whether conversion of storage capacity between flood control and conservation is warranted and, if so, the optimal storage capacity allocation. Capabilities are needed to assess system performance as precisely and meaningfully as possible for a few alternative reallocation plans rather than search through a large number of possible capacity allocations. Consequently, optimization models are not particularly advantageous for this particular application.

Inadequate basic data is a major concern in analyzing reservoir operations. Hydrologic data synthesis methods are used to overcome the limitations of short-duration records and missing data. Although rainfall, evaporation, and other data may be synthetically generated, the emphasis in reservoir operation studies is usually on extending streamflow data for reservoir inflows and flows at downstream control points.

Simulation models are often used deterministically with historical period of record or critical period inflows. However, the historical period of record is typically too short to provide an adequate basis for certain types of analyses. Stochastic hydrology techniques can generate synthetic streamflow sequences, statistically similar to the historical record, for input to simulation models. The monthly Markov model is the fundamental approach most often used for streamflow synthesis. A monthly Markov model was used in the present investigation. Wurbs, Tibbets, Cabezas, and Roy (1985) discuss alternative stochastic streamflow generation models as well as simulation and optimization modelling capabilities.

Simulation Models

The major types of simulation models typically used in analyzing reservoir operations can be categorized, as outlined in Figure 4, as (1) hydrologic, (2) economic, (3) water quality, and (4) sediment transport. Although water quality and sediment transport may be important in evaluating storage reallocation plans in some situations, in general hydrologic (water quantity) and economic analysis will be the primary thrust of the simulation effort. Consequently, the present investigation was limited to hydrologic and economic simulation models.

Hydrologic simulation models include rainfall-runoff and streamflow simulation, computation of reservoir yield and reliability, and modeling system operations for flood control and conservation purposes. Economic models typically extend hydrologic simulation to include evaluation of flood damages and benefits associated with water supply, hydropower, and possibly other conservation purposes. A specific model may contain capabilities for one or several of these types of hydrologic and economic analyses. All of the analyses are pertinent to the problem of evaluating storage reallocation plans.

Rainfall-Runoff Models

Streamflows at pertinent locations in the reservoir-stream system are fundamental input to hydrologic simulation of reservoir operations.

Historical gaged streamflow data is utilized whenever feasible. Hydrologic synthesis methods are available for extending streamflow records and filling in missing data. In many cases, streamflow records are unavailable or major changes in the watershed have rendered the historical data no longer representative of present and projected future streamflow conditions. Rainfall data combined with rainfall-runoff, or watershed, modelling are then used to develop the required streamflow data. Rainfall-runoff modelling is most often used for developing single-event flood hydrographs but can also be used to develop long-term continuous streamflow sequences. The HEC-1 Flood Hydrograph Package is an example of a single-event rainfall-runoff model which has been widely used to develop flood hydrographs for reservoir design and operation studies (Feldman, 1981). The Streamflow Synthesis and Reservoir Regulation (SSARR) Model is a continuous rainfall-runoff model developed specifically for reservoir design and operation studies (USACOE, 1976). Viessman, Knapp, Lewis, and Harbaugh (1977) provide an overview of rainfall-runoff simulation and describe a number of readily available generalized computer models. Rainfall-runoff modeling was not a major focus of the present study and is not addressed further in this report.

Streamflow Models

Streamflow modelling is an integral part of simulating reservoir flood control operations. The term streamflow model is used here to mean flood routing, water surface profile computations, and related flood wave analysis methods. Flood routing is the computation of the magnitude (discharge and/or stage) and celerity, as a function of time and location, of a flood wave propagating through a river or reservoir. Reviews of the current state-of-the-art of flood routing are provided by Fread (1982) and Wurbs (1985). Although two- and three-dimensional models have been developed, the present state-of-the-art of simulating flows in rivers and reservoirs, from a routine practical applications perspective, is one-dimensional modeling. One-dimensional flood routing models can be categorized as hydraulic, hydrologic, or purely empirical.

Hydraulic routing is based on the two one-dimensional equations of unsteady flow, commonly called the St. Venant equations, which express the physical laws of conservation of mass and momentum. Due to the mathematical complexity of the theoretical equations, for many years significant simplifications were necessary in order to obtain solutions. During the last two decades, solution of the complete St. Venant equations has become practical using numerical methods and high speed computers. A flood routing method based on the complete St. Venant equations is called a dynamic wave model, or dynamic routing. The Operational Dynamic Wave Model (DWOPER) developed by the National Weather Service is probably the most widely used of the available generalized dynamic routing models. A variety of simplified hydraulic routing techniques have been developed by ommitting or linearizing certain terms in the St. Venant equations or making other simplifying assumptions.

Hydrologic routing models are based on a relationship between storage and discharge combined with the storage form of the conservation of mass equation

$$I - O = \frac{dS}{dt}$$

where I is inflow, O is outflow, and dS/dt is change in storage with respect to time. The difference between the various hydrologic routing techniques is the form of the relationship between storage and/or outflow. Hydrologic channel routing methods include Muskingum, working R&D, variable storage coefficient, modified Puls, and their variations.

Reservoir routing is commonly performed using the modified Puls method which is based on the assumption that storage is dependent only on outflow. The above conservation of mass equation is written in finite difference form and rearranged to give the following equation

$$\frac{2S_2}{\Delta t} + O_2 = I_1 + I_2 + \frac{2S_1}{\Delta t} - O_1$$

where the subscripts 1 and 2 refer to the beginning and end of the routing interval Δt. The equation is solved step-by-step for the left-hand side, with the right-hand side of the equation known at each step of the computations. A relationship between the left-hand side of the equation and outflow must be developed from a known storage versus outflow function. A reservoir water surface elevation versus storage function is combined with the spillway and outlet works elevation versus outflow functions to obtain the required storage versus water surface elevation relationship. A level reservoir water surface is assumed.

Some flood-routing methods are based strictly on intuition and observations of past floods. Lag method and gage relations are examples of purely empirical methods.

Hydraulic routing methods compute both discharges and stages as a function of time and location. However, hydrologic and empirical routing methods are limited essentially to computing a discharge hydrograph from a known hydrograph at an upstream location. Water surface profile computations are then used to compute stages corresponding typically to peak discharges. Water surface profile computations are based on an iterative solution of the one-dimensional energy equation. The standard step method is usually used. The HEC-1 Flood Hydrograph Package and HEC-2 Water Surface Profiles computer programs are probably the most widely used generalized models for hydrologic routing and water surface profile computations. These models are used in various applications including reservoir studies.

Hydrologic routing in combination with water surface profile computations has been the traditional approach to streamflow modeling for many years. Dynamic routing is more complex but also more accurate. Dam breach flood wave analysis requirements of recent federal and state dam safety programs have provided the impetus for developing greatly expanded dynamic routing capabilities during the past decade (Wurbs 1985). Precise simulation of the effects of storage reallocation plans on major flood event stages upstream and downstream of a dam is another potential application of dynamic routing models. However, this research topic was not pursued in the present investigation. The flood routing required in the case study analysis was performed using traditional hydrologic routing methods available in the generalized computer program adopted for the study.

Reservoir Yield and Reliability

The relationship between storage capacity, yield, and reliability is a fundamental and extremely important aspect of the planning, design, and operation of a reservoir for conservation purposes. Yield is the amount of water which can be supplied from a reservoir in a specified period of time. Traditional analyses have been based on the concept of dependable or firm yield, which is the maximum rate of withdrawal which can be maintained continuously assuming the period of record historical inflows. Thus, analysis of the complex uncertainties involved in providing various levels of water supply are simplified to stating the constant yield which could be provided by a given storage capacity if future inflows reproduce the historical period of record. Reservoir inflows, as well as all other hydrologic phenomena, are stochastic in nature. Therefore, it is not possible to guarantee any yield with certainty. Reservoir reliability is an expression of the likelihood or probability of meeting given yield levels. The concept of reservoir reliability expands the concept of firm yield to provide a more meaningful basis for dealing with the uncertainties inherent in the random nature of hydrologic variables.

The yield provided by a given storage capacity is computed based on a mass balance of reservoir inflows, releases or withdrawals, evaporation and other losses, and change in storage. McMahon and Mein (1978) provide a comprehensive review of methods for analyzing reservoir capacity versus yield relationships.

Reservoir reliability is the probability that a specified demand will be met in a given future time period. Reliability is the complement of the risk of failure or probability that the demand will not be met. Reservoir reliability can be operationally defined in various ways. For example, in the present study reliability was defined as the probability of meeting demand continuously during any given year. Thus, risk of failure is the probability that a shortage would occur at least once in any given year.

Firm yield and reservoir reliability are major components of the evaluation strategy outlined in the next two chapters.

System Operation for Flood Control

Simulation of flood control operations is another major modelling task addressed by the present study. A model can include the capability to compute reservoir release rates for each time interval during the simulation period based on specified operating rules. Various forms of operating rules may be incorporated into a model. For example, when the water level is in the flood control pool, reservoir releases are typically based on emptying the pool as quickly as possible without contributing to downstream flooding. Allowable nondamaging discharges are specified at downstream control points. Reservoir inflows and incremental local inflows at the downstream control points are provided as input to the model. For each control point the model compares the discharge assuming no reservoir release to the allowable discharge. If the allowable discharge is larger, reservoir releases are made. Since the releases at the reservoir must be routed to the downstream control points to reflect attenuation and travel time, an iterative solution is required to determine the release rate which will maintain the allowable flow levels at the control points. Additional release criteria incorporated into the model includes balancing the storage levels in multiple reservoirs releasing to the same control point and limiting the rate of change of the release rate.

A generalized model developed by the Southwestern Division (SWD) of the U.S. Army Corps of Engineers is routinely used to model reservoir operations for Corps of Engineers projects in Texas and the other states in the Southwestern Division. The SWD model simulates the daily sequential regulation of a multipurpose reservoir system including the computations discussed above (Hula, 1979). As discussed later in this report, the similar HEC-5 Simulation of Flood Control and Conservation Systems computer program was used in the present study.

System Operation for Conservation

Simulation of reservoir operations for conservation purposes typically involves computing releases to meet water supply and hydroelectric power demands. Reservoir storage levels, releases, and flows at pertinent locations are computed for each time interval during the simulation. The simulation is essentially an accounting procedure for tracking the movement of water through the system. Input data includes reservoir characteristics, reservoir inflows and incremental lateral inflows at downstream control points, evaporation rates, and target demands. Diversions and return flows could occur at a reservoir or at downstream control points. Minimum instream flows may be required for fish and wildlife habitat or other purposes. HEC-5 allows diversions and instream flows to be designated as required or desired with respect to the amount of water in storage. Required demands are met as long as the reservoir storage level is above the top of the inactive pool. Desired demands are met only if the reservoir storage level is above the top of buffer pool.

Whereas flood control simulation requires a relatively short (an hour to a day) routing interval to track hydrograph peaks, simulation of conservation operations are typically based on a longer routing interval (up to a month). Flood routing techniques are not used. A simulation may be performed with historical period of record, critical period, or synthetically generated streamflows.

Economic Evaluation

Economic evaluation consists of estimating and comparing the benefits and costs, expressed in dollars, which would result from alternative plans of action. A strategy for performing an economic evaluation of storage reallocation plans is outlined in the next chapter. Fundamental economic evaluation procedures incorporated into simulation models used to analyze reservoir operations are outlined below.

Flood Damage Evaluation

Economic evaluation of flood control plans have traditionally been based on the concept of average annual damages. The inundation reduction benefit is defined as the difference in average annual damages without and with a proposed plan. Computing average annual damages using the damage-frequency method described below has been an integral part of the economic evaluation procedures followed by the Corps of Engineers and other federal agencies in planning flood control improvements for many years. The method is incorporated into several generalized computer programs and is a major component of the procedure presented in the next chapter.

Average annual damage computations are based on the statistical concept of expected value. Expected or average annual damage is computed as the integral of the damage versus exceedance probability function. Exceedance frequency versus peak discharge, discharge versus stage, and stage versus damage relationships are combined to develop the damage versus exceedance frequency function. A fundamental assumption of the procedure is that damages can be estimated as a function of peak discharge or stage. Additional analyses are required to show how damages change with variations in flow velocity, duration, and sediment content.

The magnitude of a flood threat can be quantified in various ways. Discharges, stages, and damages at specified locations can be estimated for historical storms (such as the most severe flood on record), statistical floods (such as the 50-year and 100-year recurrence interval floods), and/or hypothetical floods (such as the standard project flood). Expected or average annual damage is actually a frequency weighted sum of damage for the full range of damaging flood events and can be viewed as what might be expected to occur, on the average, in any present or future year. Additional meaningful information, including discharges, stages, and damages associated with a range of storm magnitudes, are generated in the process of computing average annual damages.

A river system is divided into reaches for analysis purposes. Average annual damages are computed for each reach and summed to get the total. Each reach is represented by an index location. The functional relationships are developed for each index location and represent the variables for the entire reach.

Since watershed and flood plain conditions change over time due to urbanization and other factors, average annual damages are computed assuming conditions expected to occur at a particular point in time. The computations can be repeated for a discrete number of future points in time. The average annual damages computed for alternative future years can be converted to an equivalent value using discounting techniques and an appropriate discount rate and period of analysis.

The basic functional relationships used in computing expected annual damages are illustrated in Figure 5. The discharge-frequency, stage-discharge, and stage-damage relationships are computed from field data. The damage-frequency relationship is derived from the other three functions. Expected annual damage is computed by numerical integration of the damage-frequency function.

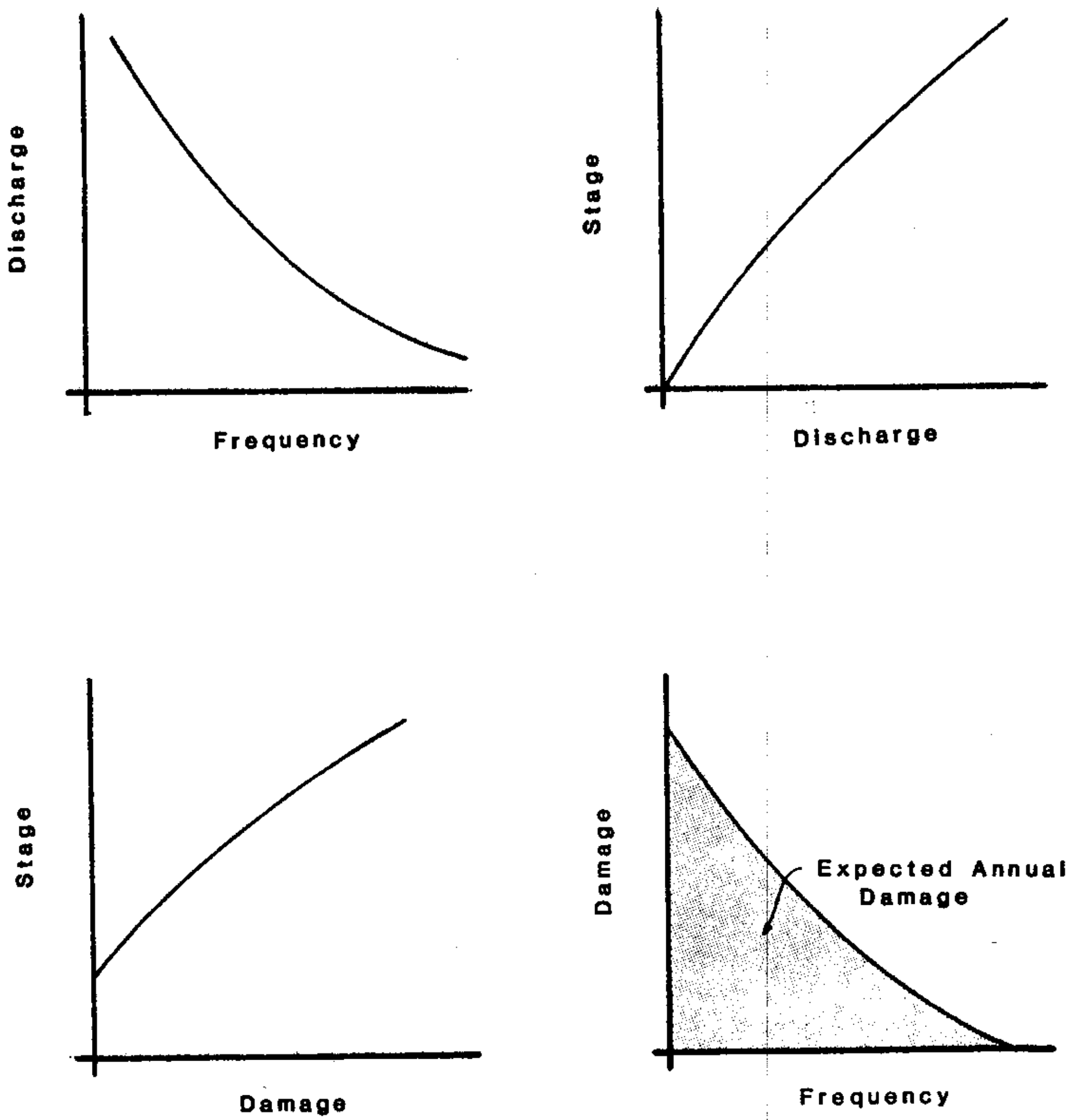
The peak discharge versus exceedance frequency relationship describes the probabilistic nature of flood flows and is developed using standard hydrologic engineering techniques. Exceedance frequency or exceedance probability is the probability that a given discharge level will be equalled or exceeded in any year. The exceedance frequency is the reciprocal of the recurrence interval. Discharge-frequency functions are commonly computed either from a statistical analysis of gaged streamflow data or through rainfall-runoff modelling.

Stage versus discharge is a basic hydraulic relationship that relates stage or water surface elevation to discharge and is commonly referred to as a rating curve. It is usually developed from water surface profile computations. A stage at an index location corresponds to a water surface profile along the river reach.

The stage versus damage relationship represents the damage, in dollars, which would occur along a river reach if flood waters reach various levels. Three alternative approaches which have been taken in developing stage versus damage relationships involve using: (1) historical flood damage data for the study area; (2) synthetic data for the study area; or (3) generalized local, regional, or national inundation depth versus percent damage functions.

A historical stage-damage curve can be developed if post-flood damage surveys have been made for several major floods which have occurred in the

Figure 5
Computation of Average Annual Damages



flood plain in the past. Damages, with price-level corrections for inflation, are plotted against stage for the historical floods. Although numerous post-flood surveys have been made at various locations, adequate historical data is still not available for most flood plains. Consequently, synthetically developed damage data or generalized depth versus percent damage functions must be used for most studies. Generalized relationships between inundation depth and damage as a percent of market value have been developed for different types of damageable property. An inventory of property located in the flood plain is combined with the generalized relationships to obtain the required stage-damage function for an index location. Synthetic damage data can be developed based on estimates of damages which would be sustained by specific activities as a result of various depths of inundation.

The effects of alternative flood damage reduction measures are reflected in the computation of the basic functional relationships. Watershed management, reservoirs, and diversions modify the frequency-discharge relationships at downstream locations. Levees, flood walls, and channel improvements change the discharge-stage function. Nonstructural measures are reflected in the stage-damage function. Any change in these three basic relationships results in a corresponding change in the frequency-damage function and thus in expected annual damages.

In order to model the effects of structural flood control improvements, a series of flood hydrographs representing a broad range of magnitudes must be routed through the stream system. Each flood provides one point on the basic relationships. Each flood consists of a set of inflow hydrographs to the stream system. Hydrographs are included on each tributary at a location upstream of all damage areas and damage reduction measures. Additional hydrographs are included at downstream locations to reflect incremental lateral inflows. The locations of the inflow hydrographs are determined based on engineering judgement considering the watershed and stream configuration and the location of damage areas and damage reduction measures. Since each flood will create one point on the frequency-damage curve which is to be numerically integrated to obtain expected annual damages, an adequate number and range of magnitude of floods are needed to properly define the frequency-damage function at each of the index damage locations.

Benefits and Losses for Conservation Purposes

Benefits for hydroelectric power can be computed by a reservoir system simulation model based on inputed primary and secondary energy values in dollars and the purchase cost for obtaining energy from an alternative source in case of a shortage in primary energy. Firm energy demands and the associated benefits are provided as input data. Secondary energy is energy in excess of firm energy which is produced by routing releases for other purposes through the turbines. Shortages are computed whenever the firm energy demands cannot be met. Cost data is provided as input for assigning dollar losses to shortages. Both the MIT Simulation Model and HEC-5 have routines for this type of hydropower economic analysis.

HEC-5 has no options for computing dollar benefits for water supply. The MIT Simulation Model allows water supply benefits and also shortage versus loss functions to be provided as input data. Economic costs associated with not meeting water supply demands are determined by the model by relating computed water shortages to the inputed shortage versus loss function.

Selected Models

Most of the computations required by the procedure outlined in Chapter 5 can be performed by existing generalized computer programs. The HEC-4, HEC-5, and MIT Simulation Model computer programs were used in the case study discussed in Chapter 6. HEC-4 was used to generate synthetic monthly streamflow data. HEC-5 was used to simulate both flood control and conservation operations, compute expected annual flood damages, and develop firm yield versus storage relationships. The MIT Simulation Model was used to compute reservoir reliability and average annual losses due to water shortages. These models are described below. Use of the models in the present study is outlined in Chapters 5 and 6.

HEC-5 Simulation of Flood Control and Conservation Systems

The "HEC-5 Simulation of Flood Control and Conservation Systems" computer program was developed and continues to be maintained by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. An initial version of the model released in 1973 has subsequently been significantly expanded. The April 1982 version used in the present study has since been superceded by a July 1985 version. The users manual (USACOE, 1982) provides detailed instructions for using the generalized computer program. Feldman (1981) describes HEC-5 as well as the several other water resources system simulation models available from the HEC.

HEC-5 simulates the operation of multipurpose, multireservoir systems. The reservoir system consists of a number of reservoirs and control points. Water demands for municipal, industrial, and/or agricultural water use, hydropower, or instream flow maintenance are specified at the reservoirs or at downstream control points. Flood control storage is operated based on flows at downstream control points. The model operates the system of reservoirs in order to best meet specified flood control and conservation requirements.

HEC-5 may be used to determine reservoir storage requirements and/or operational strategies for various water control needs. The model is also used to assist in determining reservoir releases during real-time flood control operations. Capabilities are provided for computing expected annual flood damages and hydropower benefits. A program option is also provided to determine the firm yield versus storage capacity relationship for a reservoir.

Since the program has no rainfall-runoff modeling capability, streamflows must be furnished as input data. The simulation may be performed using any one-hour or larger time interval. The time interval may vary during a simulation. For example, conservation operation is typically modeled with monthly flows, switching to daily or hourly flows for modeling operations during flood events. Flood routing methods incorporated in the program are modified Puls, Muskingum, progressive average-lag, successive average lag, and working R&D. The reservoir rule curve can vary monthly. Storage in each reservoir is discretized into levels or pools for operational control purposes. The model uses a set of operational priorities for dealing with conflicts between multiple purpose objectives and to balance storage between reservoirs.

HEC-4 Monthly Streamflow Simulation

The "HEC-4 Monthly Streamflow Simulation" program provides a variety of statistical calculations for monthly streamflow at one or several locations in a river system (USACOE, 1971 and Feldman, 1981). Generation of synthetic streamflows is accomplished based on a lag-one Markov process. Streamflows are assumed to have a log-Pearson Type III distribution.

Massachusetts Institute of Technology (MIT Simulation Model)

Strzepek and Lenton (1978) describe the MIT Simulation Model and its application to the Vardar/Axios Basin in Yugoslavia and Greece. A users manual is provided by Strzepek, Rosenberg, Goodman, Lenton, and Marks (1979). The generalized computer program provides the capability to evaluate the hydrologic and economic performance of a river basin development system. Existing and proposed reservoirs, hydroelectric power plants, thermal power stations, irrigation areas, and diversions and withdrawals for municipal, industrial, and other uses are represented in the model as a system of arcs and nodes. The model computes the monthly flows at all nodes in the basin, given the streamflows at the start nodes. System reliability in meeting water demands is assessed. Irrigation, hydroelectric power, and municipal and industrial water supply benefits are computed and compared with project costs. Benefits are divided into long-term benefits and short-term losses.

CHAPTER 5
PROCEDURE FOR EVALUATING STORAGE REALLOCATION PLANS

The research included development of a procedure for hydrologic and economic evaluation of plans for reallocating storage capacity in an existing reservoir between flood control and conservation purposes. The procedure is based on the concept that risks and economic consequences are associated with both failing to meet various levels of water demand and failing to prevent various levels of flooding. The purpose of the procedure is to estimate and display the risks and consequences associated with alternative storage capacity allocations in such a manner as to provide meaningful information for use in the decision making process. The procedure is outlined in this chapter. The results of applying the procedure to a case study are presented in the following chapter.

Relationship of Proposed Procedure to Traditional Evaluation Methods

In the past, flood control and conservation operations have been handled as separate activities without a need for comparison of tradeoffs and interactions. Traditional design philosophy and analysis methods for sizing storage capacities and establishing operating policies for conservation purposes are totally different from those associated with flood control. The research literature as well as practices and procedures followed by the water resources development agencies have not significantly addressed the problem of evaluating tradeoffs and interactions between using limited available storage capacity for flood control versus conservation purposes.

Flooding has traditionally been associated with the concept of risks (probabilities) and consequences (damages). The federal water agencies have developed detailed methods for quantifying the economic benefits associated with reducing the risk of flooding. These methods were incorporated into the present research. The methodology developed is consistent with traditional flood control planning practices and procedures. On the other hand, water supply planning has been based primarily on assuring that firm yield exceeds demands. Little attention has been given to quantifying risks and consequences in water supply planning and management. A major thrust of the present research was to develop expanded capabilities to evaluate the risks and consequences of failing to meet various levels of water demand. Water supply was treated in a manner analogous to flood control. The objective is to quantify the risks and consequences associated with both water supply and flood control in such a manner that the impacts of a storage reallocation on the two different project purposes can be meaningfully compared.

Flood Control

One of the major accomplishments of the federal program for water resources development has been the introduction of economic criteria into government decision making. The Flood Control Act of 1936 initiated the use of benefit-cost analysis as a basis for evaluation of proposed federal flood control projects. Since 1936, economic evaluation procedures have been developed and refined along with their application in the planning of essentially all federal flood control projects.

Flood control benefits are derived from reducing damages and permitting more efficient use of land resources. Benefits are conceptually considered in three categories: inundation reduction, intensification, and location (Water Resources Council, 1983). Inundation reduction benefits apply in a situation in which an activity uses the flood plain exactly the same with and without a flood damage reduction plan. The benefit is the increase in net income to the flood plain activity. Intensification benefits occur when a commercial, industrial, or agricultural activity in the flood plain modifies its operation because the reduction in flood damages makes it profitable to do so. Location benefits occur when an activity uses the flood plain with a flood control project but uses a site out of the flood plain if there is not a flood control project. The location and intensification benefits are the increased net income to the activity and land owner comparing the method of operation without a flood control project to that with a project.

The estimation of inundation reduction benefits attributable to reducing physical flood damages is the heart of the economic evaluation process. This benefit is the difference in average annual damages with and without a proposed plan. The damage-frequency method described in the previous chapter is the standard procedure for estimating average annual damages. This method was adopted in the present investigation.

Intensification and location benefits are considered to typically be relatively insignificant in evaluating storage reallocation plans and were not included in the investigation. In the case study, present and potential future flood plain occupants were not expected to modify their activities in response to a storage capacity reallocation. However, intensification and location benefits could be significant in certain situations. These benefits would then be added to inundation reduction benefits in a storage reallocation study similarly to any other type of flood control planning study.

Water Supply

Conservation storage capacity for the reservoirs in Texas, and elsewhere, were established and continue to be viewed in terms of firm yield as determined using a historical period of record analysis. The water supply planning and management philosophy has been simply that firm yield should exceed water demands. In federal planning of multipurpose reservoir projects, water supply benefits are estimated as the cost of the least costly alternative means of providing the same quantity and quality of water assuming the proposed project is not implemented. The major policy emphasis in recent years on demand management and achieving more efficient water use has resulted in reservoir planning studies now including projections of water needs alternatively assuming reasonable demand management strategies are and are not adopted. However, the probabilities associated with not meeting various demand levels and economic consequences of failing to meet demands typically are not being addressed in the actual planning, design, and operation of reservoir projects.

Reservoir reliability analysis provides estimates of the probabilities of failing to meet demands. Wurbs, Tibbets, Cabezas, and Roy (1985) summarize the current state-of-the-art of analyzing reservoir reliability and associated synthetic streamflow generation methods. The significant gap between the extensive research literature that has developed in recent decades and the practices followed by the various entities which are actually responsible for constructing and operating reservoir projects is noted. The present study did not attempt to develop additional new techniques for analyzing reservoir reliability. Very fundamental reservoir reliability and synthetic streamflow generation techniques are incorporated into the methodology. However, the storage reallocation problem does represent a significant new application for reliability analysis.

A key component of the present research was developing an approach for estimating the consequences of failing to meet municipal and industrial water demands in terms of economic losses. The procedure developed includes: formulation of emergency demand management and supply augmentation strategies; estimating costs for implementing the emergency measures and other losses to water users; and computation of average annual losses. Economic consequences of failures to meet water demands have traditionally not been quantified in planning, design, and operation of reservoirs.

Storage Reallocation Studies

The few reallocation studies performed to date have incorporated the traditional procedures used in regular planning and design studies. Firm yield is computed for the alternative water supply allocations. The flood storage capacity is evaluated in terms of the return interval of a design flood that can be contained without damaging releases. The present research incorporated these types of analysis but also provided additional expanded capabilities for quantifying risks and consequences and comparing tradeoffs between flood control and water supply.

Description of Procedure

The general procedure outlined here could actually be adapted to a broad range of applications related to optimization of multipurpose reservoir operations. However, the case study and the present discussion focus specifically on the following storage reallocation problem. The total storage capacity in the reservoir is assumed fixed. Raising the dam or major structural modifications to the spillway or outlet works are not a part of the proposed reallocation plans. Alternative storage capacity allocations between flood control and municipal and industrial water supply purposes are defined by a designated top of conservation (bottom of flood control) pool elevation. The flood control release schedule remains constant for the alternative storage capacity allocations. The decision problem is to determine the optimum top of conservation pool elevation which maximizes the overall beneficial use of the limited available storage capacity.

For most multipurpose reservoirs in Texas, the top of conservation pool is constant without seasonal variation. The present study also investigated the potential for seasonal rule curve operation in which the designated top of conservation pool elevation varies during the year.

The risks of failing to prevent flooding and failing to meet water demands will change in response to a reallocation of storage capacity. Depending on the magnitude of the storage reallocation, the changes in risk may or may not be significant enough to meaningfully quantify. The proposed methodology is based on estimating and displaying the risks and consequences associated with alternative storage allocations. The objective is to develop an understanding of system performance which will provide an objective basis for determining optimal storage allocations.

The proposed procedure is actually a general evaluation approach incorporating several hydrologic and economic analysis methods. A major thrust of the approach is to determine the storage capacity allocation which will minimize the total economic losses due to flooding and water shortages. Raising the top of conservation pool elevation increases the losses due to flooding and decreases the losses due to water shortages. For given conditions of water demand and flood plain development, the top of conservation pool elevation which minimizes losses plus implementation costs is determined. The bulk of the analysis effort is directed toward estimating average annual economic losses, in dollars, due to flooding and water shortages. The estimated costs of implementing storage reallocation plans are combined with the losses due to failures to prevent flooding and meet water demands to determine the economic feasibility of the alternative plans. Discounting techniques are required such that implementation costs and losses are on an equivalent time basis.

The economic evaluation provides both an economic feasibility criterion and an objective function. A reallocation plan is economically feasible if the reduction of flooding and water shortage losses exceeds the costs to be incurred in implementing the plan. The economic objective function is minimization of total costs which include losses due to flooding, losses due to water shortages, and implementation costs. The economic evaluation can significantly contribute toward providing quantitative information for use in the decision making process. However, the numerous complexities and uncertainties inherent in evaluating proposed storage reallocation plans dictates development of as thorough an understanding of system performance as practical. Consequently, additional hydrologic analyses are included in the overall methodology because the information provided helps develop a better understanding of how the reservoir effectiveness changes with reallocations of storage capacity. The overall procedure is summarized below following the outline presented in Figure 6.

Flood Control

As indicated in Figure 6, two general types of analyses are included in the procedure for evaluating the impacts of storage reallocations on the flood control effectiveness of a resevoir: (1) routing statistical and hypothetical inflow hydrographs through the reservoir and (2) economic flood damage evaluation. The first analysis does not contribute to the economic evaluation but does provide meaningful additional information. The second analysis provides

Figure 6

Procedure for Evaluating Storage Reallocation Plans

I. Flood Control

   A. Evaluation of storage capacity in terms of hydrologic effects on statistical and hypothetical reservoir inflow hydrographs

   B. Evaluation of flood damages

      1. Damages for historical storms

      2. Expected annual damages

II. Water Supply

   A. Water demand study

      1. Present water use

      2. Long-term demand management plans

      3. Projections of future water needs

   B. Hydrologic analysis

      1. System simulation

      2. Firm yield versus storage capacity relationship

      3. Reliability versus storage capacity and demand relationships

   C. Estimation of economic losses due to water shortages

      1. Development of shortage versus loss relationship

         a. Formulation of emergency demand management and supply augmentation plans

         b. Estimation of effectiveness and implementation costs for emergency measures

         c. Estimation of other losses to water users

      2. Computation of average annual losses

III. Implementation Costs for Storage Reallocation Plans

IV. Comparison of Analysis Results for Alternative Storage Allocations the average annual flood losses which are directly compared to water shortage losses to evaluate the economic tradeoffs between flood control and water supply.

Inflow hydrographs associated with a range of flood frequencies or return intervals can be routed through the reservoir for each alternative flood control storage capacity being considered. Reservoir flood control effectiveness can then be expressed in terms of the return interval of the largest flood that can be contained without exceeding the top of flood control pool. Peak reservoir water surface elevations and outflows for a range of floods can also be displayed. Hypothetical floods such as the probable maximum flood can likewise be routed through the reservoir to analyze the impact of storage reallocations on extreme events. The reservoir is analyzed independently of downstream flooding conditions or other reservoirs in the system. The reservoir water surface is assumed to be at the top of conservation pool at the beginning of each flood event. A simplified approach based on the unit hydrograph concept was used to develop the reservoir inflow hydrographs for the case study. Other traditional hydrologic engineering methods could be used as well. In the case study, the routing computations were done manually using the release schedule curves provided in the regulation manual to compute the outflow for the current inflow rate and water surface elevation.

Economic flood damage evaluation consists primarily of computing average annual damages. Statistical frequency versus damage relationships and the damages resulting from individual historical flood events can also be displayed. Average annual damage computations are described in the previous chapter. The computer program HEC-5 was used in the case study to compute average annual damages.

Water Supply

The evaluation of the water supply aspects of alternative storage allocations includes estimation of water demands, hydrologic simulation, and estimation of economic losses due to water shortages.

Water demand is the key factor in determining whether storage capacity reallocations are justified. The relationships between storage capacity and reliability and between storage capacity and economic losses are developed for a given demand. Present demands are determined based on water use records. Future needs are projected using standard techniques based on population projections, per capita water use rates, and other economic indicators.

Alternative demand projections are developed with and without assumed implementation of long-term demand management strategies. In general, a study can be complicated by the potential for demands to be met by alternative sources of supply other than the reservoir system being considered. However, in the case study, the demands to be met by the reservoir under consideration could be clearly delineated. The case study relied upon water needs projections from a Corps of Engineers water supply study. Monthly water use variations were determined from an analysis of water use records.

Development of relationships between reliability, yield, and storage capacity through hydrologic simulation is accomplished concurrently with computing water shortages for the economic evaluation. Reservoir reliability is defined here on an annual basis as the percentage of the total years simulated in which target demands are fully met without a shortage. The risk of failure is the complement of reliability or the percentage of the total years simulated for which demands could not be fully met. Hydrologic simulation results are also meaningfully displayed by plotting storage levels over a period of record simulation or by developing a frequency versus minimum storage level relationship for each alternative water supply capacity. The traditional firm yield versus storage capacity curve also provides meaningful information even though not used directly in the economic evaluation. In the case study, the MIT Simulation Model was used to compute reservoir reliability for a range of alternative demands and storage capacities. HEC-5 was also used to simulate the water supply operations and develop a firm yield versus storage capacity curve. The simulations were performed alternatively with historical period of record monthly reservoir inflows and a much longer sequence of monthly inflows synthetically generated using HEC-4.

Average annual economic losses due to water shortages are computed based on a functional relationship between water shortage as a percentage of demand in a given year and economic losses in dollars. The shortage versus loss function is developed for a specified water demand level. The reservoir operation is simulated using a historical or synthetically generated sequence of monthly streamflows, monthly evaporation rates, specified target demand level, and reservoir storage capacity. As long as adequate water is available, the target demands are met each month. The simulation model computes the shortage for each month in which the demand is not met due to the reservoir being empty. The monthly shortages (in million gallons) are summed to obtain the total shortage for the calendar year. The yearly shortage expressed as a percentage of demand is combined with the shortage versus loss function to obtain the loss in dollars for that year. The yearly losses over the simulation period are summed and divided by the total number of years to obtain the average annual economic loss. The MIT Simulation Model was used in the case study to perform the computations.

The shortage versus loss function is developed and applied on a yearly basis instead of monthly based on the following premise. Water supply managers will likely take action to reduce demands on a reservoir before the reservoir is completely depleted. The emergency action may prevent the reservoir from actually emptying. Also, instead of the reservoir being completely empty during one or several months during the dry season of a year, the shortage will be spread out over the year by emergency demand management or supply augmentation measures. The emergency measures are implemented for a single drought situation during the year rather than several times representing several different months.

The MIT Simulation Model computes economic losses concurrently with reservoir reliability. Reliability is based on counting the number of years in which target demands are not met. Losses are based on tabulating the volume of the shortages in each year.

The key aspect of estimating average annual losses is the development of the shortage versus loss relationship. This planning task is the major focus of the dissertation by Cabezas. The procedure is based largely upon professional judgement, the limited data and experiences reported in the literature on demand management and coping with drought conditions, and an understanding of the particular community or region being investigated. Alternative emergency demand management and supply augmentation strategies are formulated. The emergency management strategy changes as the water shortage becomes more severe. For example, the strategy could evolve from encouraging voluntary demand reductions to mandatory rationing to importing water by trucks or temporary pipelines. The effectiveness in reducing demands or augmenting supplies and implementation costs are estimated. Additional costs and losses incurred by water users due to shortages are also estimated. The resulting functional relationship between the total volume shortage of water occurring during a year and the resulting dollar costs and losses is provided as input data to the MIT Simulation Model for use in the computations described above.

Implementation Costs

Engineering cost estimates are required for relocation or alteration of any facilities necessitated by raising or lowering the top of conservation pool. This could include boat ramps and other recreational facilities, roads and bridges, and water supply intake structures.

Comparison of Analysis Results for Alternative Storage Capacity Allocations

The purpose of the procedure is to evaluate whether a reallocation between flood control and municipal and industrial water supply is warranted and, if so, to determine the optimum allocation. Alternative allocations are defined by the designated top of conservation pool elevation, which sets the flood control and water supply storage capacities. Seasonal rule curve operation plans may be included with the alternative storage capacity allocations. The computations described above are repeated for as many discrete alternative allocations as necessary to develop an understanding of system response to reallocations and to determine an optimum allocation.

Water demands and flood plain development change over time. Likewise, reservoir storage capacity decreases due to sedimentation. Consequently, the computations are performed for conditions at a particular point in time. The computations may be repeated for different conditions, such as 1990, 2000, 2010, etc. In the case study, water demands increased significantly over time, but flood plain conditions were assumed to remain constant. To simplify the analysis, a single future reservoir storage capacity versus elevation relationship was used for all time periods which included an estimated 50-year sediment accumulation.

Water shortage losses, flood losses, and implementation costs are converted to a common time basis for purposes of comparison. Standard equivalence formulas are used with a selected period of analysis and discount rate.

Demand management strategies are categorized as long-term or emergency. Long-term measures are reflected in the water demands. Emergency measures are reflected in the shortage versus loss function.

As previously discussed, a storage reallocation plan is economically warranted if the reduction in total flood and water shortage losses exceeds implementation costs. The economic objective function for determining the optimum storage capacity allocation is to minimize total cost which is the sum of flood losses, losses due to water shortages, and implementation costs.

The merit and feasibility of storage capacity reallocation proposals involves complex institutional, financial, legal, political, and public opinion considerations as well as the hydrologic and economic factors addressed here. The hydrologic and economic evaluation methodology involves approximations and modelling uncertainties. The methodology is intended to provide an understanding of system performance and meaningful quantitative information for use in the public decision making process but not stringent criteria to be rigidly followed.

CHAPTER 6
CASE STUDY

The procedure outlined in the previous chapter was developed and tested by application to an evaluation of a proposed storage capacity reallocation in Waco Reservoir. This particular reservoir was selected as a case study partially because the Corps of Engineers, at the request of the Brazos River Authority and City of Waco, recently studied and recommended a storage reallocation. Waco Lake also provides a good case study because it is located in central Texas and has physical and hydrologic characteristics and operating procedures representative of typical multipurpose reservoirs in the state. Waco Lake and the Corps of Engineers reallocation study are briefly described and the results of the present evaluation summarized in this chapter. The analysis results are documented in detail by the dissertation by Cabezas and thesis by Tibbets.

Description of the Case Study Reservoir

The Waco Dam and Reservoir project was authorized by the Flood Control Act of 1954. Construction was initiated in 1956, and deliberate impoundment began in February 1965. The dam and reservoir are located entirely within the corporate limits of the City of Waco in central Texas. The dam is on the Bosque River 4.6 miles above its confluence with the Brazos River. At the top of conservation pool, the reservoir inundates the confluences of the four major tributaries of the Bosque River: North Bosque, Hog Creek, Middle Bosque, and South Bosque. The reservoir has a drainage area of 1,670 square miles. The water surface area at top of conservation pool is 7,270 acres.

Waco Dam is 24,620 feet long with a maximum height of 140 feet. The dam is an earthen embankment except for a 1,034 foot concrete gravity spillway section. The spillway is controlled by fourteen 40-feet x 35-feet tainter gates. The outlet works consists of a 20-foot diameter conduit controlled with Broome-type tractor sluice gates. Pertinent elevations in feet above mean sea level are as follows: streambed, 370 feet; top of conservation pool, 455 feet; spillway crest, 465 feet; top of tainter gates, 500 feet; maximum design water surface, 505 feet; and top of dam, 510 feet.

Project purposes are flood control, municipal and industrial water supply, and recreation. Flood control, conservation, and sediment reserve capacities are 553,300 acre-feet, 104,100 acre-feet, and 69,000 acre-feet. The 69,000 acre-feet of sediment reserve was available at the time of initial impoundment to provide for 50 years of sedimentation. The Fort Worth District of the U.S. Army Corps of Engineers constructed, owns, and operates the project. Releases from the conservation pool are made at the discretion of the local project sponsors. The city of Waco and the Brazos River Authority (BRA) have contracted with the Corps of Engineers for 12.6 percent and 87.4 percent, respectively, of the conservation storage. The BRA has contracted with the city of Waco to supply the city water from BRA's 87.4 percent share of the conservation pool. Thus, all of the conservation storage in Waco Lake is committed for providing municipal and industrial water supply for the city of Waco and its suburbs.

The Corps of Engineers operates the project. Water supply releases are made as requested by the city of Waco to meet demands. Normally no flood-control releases are made if the reservoir level is at or below the top of conservation pool, elevation 455.0. However, if flood forecasts indicate that the inflow volume will exceed the available conservation storage, flood control releases may be made if downstream conditions permit. Whenever runoff-producing rainfall occurs or a flood is in progress on the Bosque and Brazos Rivers and the reservoir level is in the flood control pool, all of the gates are closed. The gates remain closed until the flow on the Bosque and Brazos Rivers has crested and receded to 50,000 cfs on the Bosque River at the Waco gage and 60,000 cfs on the Brazos River at the Waco and Richmond gages. The flood control pool is emptied as quickly as possible without exceeding these allowable downstream flow rates unless the schedule shown in Figure 3 indicates a larger release. The Figure 3 schedule controls during extreme flood events.

Waco Lake is a component of an eleven reservoir system operated by the Corps of Engineers to control flooding in the Brazos River Basin. The reservoirs are operated to maintain allowable discharges at a number of downstream control points, several of which are common to two or more reservoirs. In making releases to common control points, system operation is based on balancing the percentage full of the flood control pools in each reservoir. Waco Reservoir is operated primarily in conjunction with Whitney Reservoir which is located on the Brazos River 18 miles upstream of the Bosque River confluence. The Brazos River Basin has a drainage area of 45,570 square miles of which 1,650 acres are above Waco Dam. The Richmond gage, which serves as the most downstream control point for Waco and the other reservoirs, is over 200 river miles downstream of Waco Dam.

Fort Worth District Reallocation Study

In March 1979, the Brazos River Authority, in cooperation with the City of Waco, requested that the Fort Worth District (FWD) investigate the feasibility of increasing the conservation storage capacity in Waco Reservoir to provide a greater dependable water supply yield. A subsequent study by the FWD resulted in a recommendation that 47,500 acre-feet or 8.6 percent of the flood control capacity be reallocated to water supply. The reallocation would raise the top of conservation pool from elevation 455.0 feet above mean sea level to about 462.0 feet. The dependable yield of the reservoir would be increased from 54.9 mgd to about 70.0 mgd. A loss of 47,500 acre-feet of flood control capacity was estimated to reduce protection from a 100-year to about an 80-year recurrence interval design flood (USACOE, 1982).

The proposed reallocation was approved by the Office of the Chief of Engineers in April 1983. The Chief of Engineers, located in Washington, D.C., has the discretionary authority to approve reallocations of not greater than 15 percent of the total storage capacity allocated to all authorized federal purposes or 50,000 acre-feet, whichever is less. Larger storage capacity reallocations in federal projects would require Congressional approval.

A contract between the Brazos River Authority (BRA) and the federal government for the Waco Reservoir reallocation was executed in September 1984. The contract provides for the BRA to reimburse the cost for relocating recreation facilities plus the allocated value of the water supply storage. The next step in the process is for BRA to provide funds in an escrow account. The FWD will then relocate the recreation facilities as required and impound water in accordance with the raised top of conservation pool elevation.

The FWD reallocation study report includes: an analysis of present and projected future water demands; formulation of alternative strategies for meeting the water needs; selection of the storage reallocation plan; environmental impact assessment; cost estimate for implementing the recommended plan; allocation of costs between the federal government and Brazos River Authority; and a draft contract between the federal government and Brazos River Authority for use of the additional conservation storage and repayment of associated costs (USACOE, 1982). The effects on reservoir performance of the proposed capacity reallocation were evalauted primarily in terms of firm yield and the recurrence interval of the design flood which could be contained by the flood control pool.

Application of Proposed Procedure

The present study included an evaluation of the proposed storage reallocation in Waco Lake using the procedure described in the previous chapter. The outline presented in Figure 6 is followed here in presenting the results of the case study analysis.

The data required to perform the evaluation were obtained primarily from documents and unpublished files provided by the Fort Worth District (FWD) office of the Corps of Engineers. The Waco Lake Regulation Manual was the source for much of the data, including: physical characteristics of the reservoir; operating procedures; monthly streamflows at the damsite for the period 1907-1970; average monthly net evaporation rates; reservoir inflow unit hydrograph; and probable maximum flood inflow hydrograph. Likewise, information required for Whitney Lake was obtained from the Whitney Lake Regulation Manual. Hydrologic records for the two reservoirs, including daily inflows, were furnished by the FWD Reservoir Control Section. U.S. Geological Survey streamflow records provided daily flows at six downstream gaging stations. Channel routing coefficients were taken from previous FWD studies. Discharge versus damage curves were also provided by the FWD from unpublished files. Present and projected future water demands were available from the FWD Waco Lake Reallocation Study. Water use data were also obtained from the City of Waco.

Flood Control

As indicated in Figure 6, the flood control analysis consists of (1) evaluation of the hydrologic effects of the reservoir on statistical and hypothetical reservoir inflow hydrographs and (2) evaluation of flood damages.

Reservoir Effects on Statistical and Hypothetical Inflow Hydrographs

The following simplified approach was followed in developing statistical inflow hydrographs. A 46-year annual series of peak discharges was assembled from several sources and fitted to a log Pearson Type III probability distribution to develop a peak discharge versus exceedence frequency function. The reservoir inflow unit hydrograph was obtained from the Waco Lake Regulation Manual. For a given exceedence frequency, the ratio of peak discharge divided by unit hydrograph peak discharge was computed. This ratio was then multiplied by the unit hydrograph ordinates to obtain the reservoir inflow hydrograph associated with the specified exceedence frequency.

The inflow hydrographs were routed through the reservoir manually using the regulation schedule shown in Figure 3 to determine the release rate for a given reservoir water surface elevation and inflow rate. The water surface was assumed to be at the top of conservation pool at the beginning of each flood. The spillway and outlet works gates were assumed to remain closed until releases were indicated by the Figure 3 regulation schedule. Thus, the computed peak reservoir water surface elevations and outflows are independent of downstream flooding conditions. The probable maximum flood taken from the Regulation Manual was routed through the reservoir by the same procedure. The results of a series of routings for several assumed top of conservation pool elevations are presented in Tables 4 and 5.

The flood control storage capacity was also quantified in terms of the exceedence frequency or recurrence interval of a design flood which just fills the flood control pool without overflowing. This is the return interval of the reservoir inflow hydrograph which has a total runoff volume equal to the flood control storage capacity of the reservoir. The design recurrence interval, as thus defined, is tabulated in Table 6 for a range of alternative storage allocations.

As illustrated in Table 4, a storage capacity reallocation has little effect on peak outflows for smaller floods which are contained without exceeding the flood control capacity and on extreme events approaching the probable maximum flood (PMF). Table 4 shows that raising the top of conservation pool elevation from 455 to 470 feet increases the peak outflow for the 200-year flood from 98,000 cfs to 224,000 cfs. As indicated in Table 6, with the top of conservation pool at elevation 455 feet, the 109-year flood just fills the flood control pool. Raising the top of conservation pool to elevation 470 feet reduces the flood control capacity from a 109-year to a 57-year recurrence interval design flood. Although this type of information is not used in the economic evaluation, it does significantly contribute toward meaningfully displaying the hydrologic impacts of storage reallocations.

Evaluation of Flood Damages

Expected annual damages were computed with HEC-5 using hydrographs from seven historical floods. Five damage index locations were used. The damage index locations were located at stream gaging stations which also served as control points for computing incremental lateral inflows. The damage index locations are all on the Brazos River at distances ranging from 5 to over 200

Table 4
Peak Outflows for Statistical and PMF Inflow Hydrographs

Return Interval (years)	Peak Inflow (1000 cfs)	Peak Outflow (1,000 cfs) for Top of Conservation Pool Elevation (feet) 455	462	470
50	160	0	0	0
75	187	0	0	30
100	208	0	18	74
125	226	14	44	111
150	241	36	100	148
200	264	98	130	224
250	284	100	182	248
300	300	174	198	260
400	327	188	252	280
PMF	623	570	572	574

Table 5
Peak Water Surface Elevations for
Statistical and PMF Inflow Hydrographs

Return Interval (years)	Peak Water Surface Elevation for Top of Conservation Pool Elevation (feet) 455	462	470
25	485	488	492
50	492	494	498
75	494	497	500
100	497	500	501
125	500	501	501
150	500	501	501
200	501	501	501
250	501	501	501
300	501	501	501
400	501	501	501
PMF	504	504	504

Table 6
Nondischarging Design Return Intervals

Top of Conservation Pool Elevation (feet)	Water Supply Capacity (acre-feet)	Flood Control Capacity (acre-feet)	Design Return Interval (years)
447	62,000	595,400	113
451	80,000	577,400	111
455	104,100	553,300	109
462	147,500	509,900	84
470	220,000	437,400	57

Table 7
Average Annual Damages

Top of Conservation Pool Elevation (feet msl)	Average Annual Damages ($1,000)	Change in AAD ($1,000)
447	4,219	
		109
451	4,266	
		62
455	4,328	
		63
462	4,391	
		270
470	4,598	

river miles below Waco Dam. Whitney and Waco reservoirs were both operated in the simulation.

Exceedence frequency versus peak discharge functions were developed for each control point by fitting a 18-year annual series to a gumbel probability distribution at each gaging station (control point). Most of the gaging stations have longer periods of record but construction of Waco Reservoir and other projects render the data nonhomogeneous. The short length of record of the data limits the accuracy of the frequency versus discharge relationships. Although rainfall-runoff modelling could be used to develop better frequency versus discharge functions, the required effort was considered to not be warranted for the present study.

Discharge versus damage curves for each index location were obtained from the Fort Worth District (FWD). These relationships are based on field studies conducted during the early 1960s in conjunction with planning for construction of the project. Consequently, the estimates of damage susceptibility are outdated and very approximate but still considered to be adequate for purposes of the present study. The effort required to conduct a new field damage survey was clearly outside the scope of the study. Most of the damage susceptibility is related to agriculture. A recent field examination of the flood plain through the City of Waco by FWD personnel in conjunction with the FWD reallocation study indicated very little urbanization had occurred. Price level indices were used to update the damage data for inflation.

The average annual damages computed for four alternative storage capacity allocations are tabulated in Table 7. Reallocation of storage capacity in Waco Reservoir is indicated to have relatively little effect on the total average annual damages experienced in the Brazos River Basin. Raising the top of conservation pool elevation from 455 to 470 feet increases the average annual damages from $4,328,000 to $4,598,000, for an increase of $270,000. This relatively small change is due to over 96 percent of the damages occurring at locations greater than 200 miles below Waco Dam. The influence of a flood control reservoir naturally decreases with distance downstream. Waco Reservoir is on a tributary which represents a small portion of the Brazos River drainage area. Also, raising the top of conservation pool from 455 to 470 feet results in a decrease from 100-year to about 56-year in the recurrence interval of an inflow hydrograph which can be contained by the flood control pool. Therefore, floods with a recurrence interval of less than about 56-years are not affected at all by the reallocation.

Water Supply

The evaluation of the water supply aspects of alternative storage allocations consists of estimation of water demands and hydrologic and economic analysis of the effectiveness of the reservoir in meeting the demands.

Water Demand Study

The water supply study area consists of the city of Waco and nearby cities of Woodway, Hewitt, Robinson, and Bellmead. Population projections for the five cities are tabulated in Table 8. The city of Waco supplies water to about 32,000 municipal and industrial customers and accounts for approximately 90 percent of the water use in the study area. Waco Reservoir and groundwater are the current source of supply for the study area. However, groundwater availability is limited and rapidly declining. The cities of Woodway, Hewitt, Robinson, and Bellmead currently rely primarily on groundwater but are expected to need an alternative source by 1990. The present study is based on the premise of all five cities relying solely on Waco Reservoir.

Information on present and future water needs were obtained from the FWD reallocation report. Water use records were also obtained from the city of Waco. The FWD reallocation report provides water use projections for the 5-city study area with and without assumed implementation of long-term demand management measures. The projections without future adoption of demand management measures are tabulated in Table 9. Figure 7 shows projections for both with and without demand management conditions. Water use records obtained from the city of Waco were analyzed to obtain the monthly water use variations shown in Table 10.

Hydrologic Analysis

The hydrologic simulation studies included: (1) period of record simulations for alternative demand levels and conservation capacities, (2) analysis of firm yield, and (3) analysis of reliability. The reservoir reliability analysis included alternatively using the historical period of record and a much longer sequence of synthetically generated streamflow data.

System Simulation. Monthly storage levels were computed with HEC-5 for alternative demands and conservation capacities given 916 months of historical average monthly streamflow, average evaporation rates for each month of the year, and factors representing the proportion of the annual demand occurring in each month of the year. The storage levels resulting from an operating policy consisting of a top of conservation pool elevation of 455 feet and

Table 8
Population Projections
Waco Standard Metropolitan Statistical Area

City	Population for Year				
	1980	1990	2000	2010	2040
Waco	105,800	110,600	115,400	120,200	134,600
Woodway	7,900	10,400	12,900	15,400	17,000
Hewitt	5,000	8,000	10,000	11,600	15,000
Robinson	6,600	8,300	10,200	12,000	17,400
Bellmead	9,300	10,200	11,500	12,800	16,000
Total	168,600	187,300	194,000	206,800	252,000

Source: U.S. Army Corps of Engineers, Fort Worth District, Waco Lake Storage Reallocation Study, October 1982.

Table 9
Water Demands

Year	Water Demand in MGD		
	Municipal	Industrial	Total
1980	28.1	6.0	34.1
1990	34.6	8.7	43.3
2000	39.1	12.3	51.4
2010	42.0	16.0	58.0
2020	45.0	19.8	64.8
2030	47.0	24.6	71.6
2040	49.2	30.2	79.4

Table 10
Monthly Demands

Month	Monthly Water Use as a Percentage of Annual Water Use
January	6.58
February	6.16
March	6.41
April	6.96
May	7.93
June	9.55
July	11.51
August	11.68
September	10.29
October	8.53
November	7.27
December	7.13

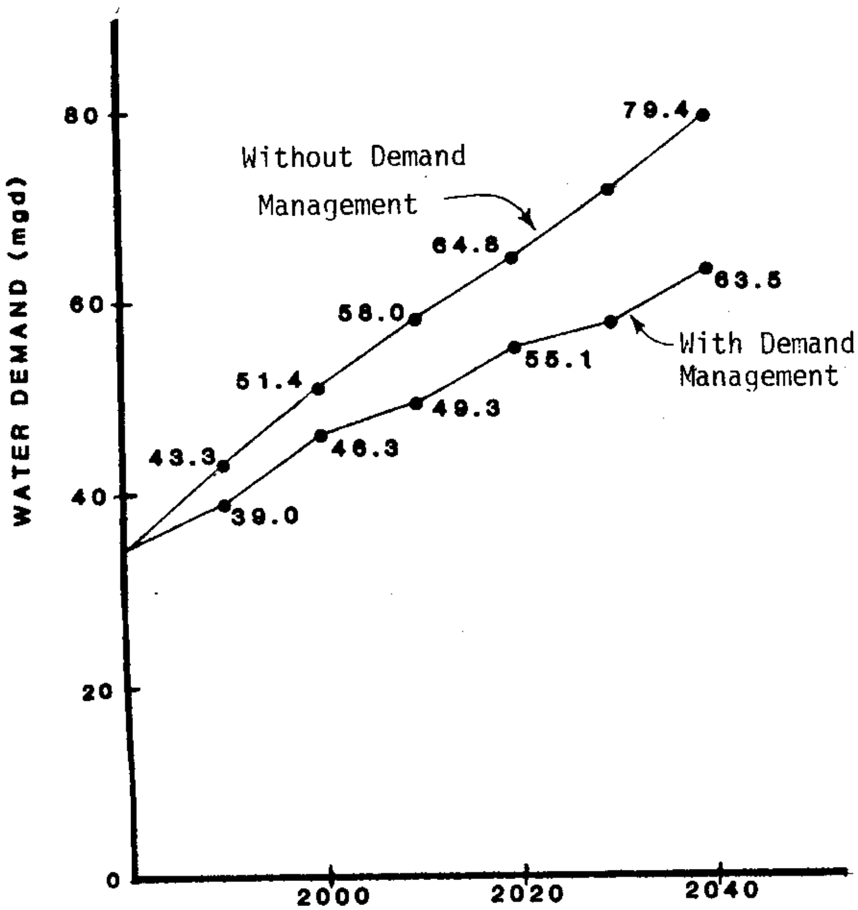


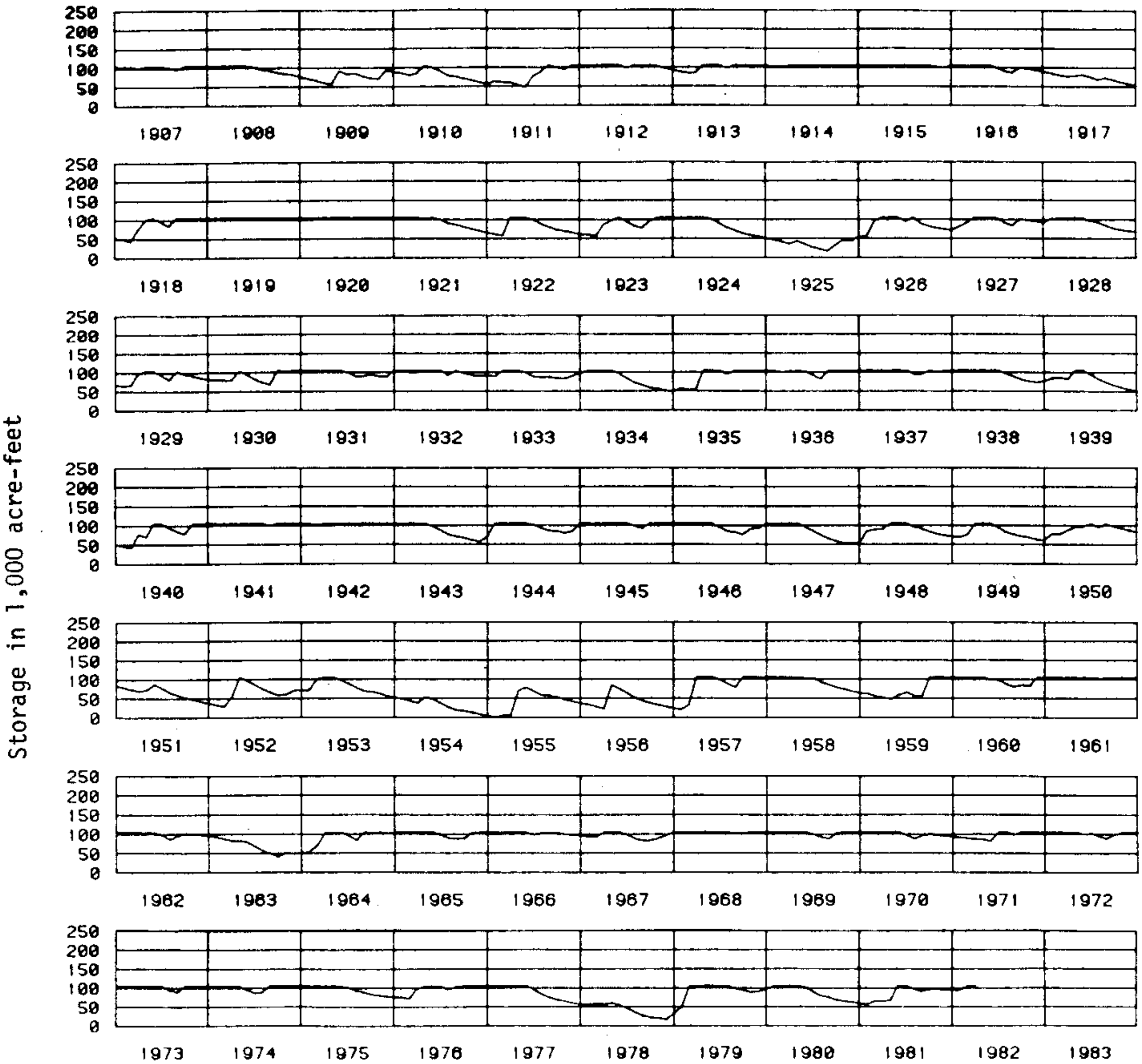
Figure 7
Water Demands

constant yield of 52.3 mgd are plotted in Figure 8. This is the present conservation capacity with the demand set equal to firm yield.

Firm Yield. Firm yield versus storage capacity relationships were computed with HEC-5 based on the historical record of monthly streamflow. The sensitivity of firm yield to various assumptions regarding key input data is displayed in Figure 9. Five firm yield versus storage capacity curves are shown. The base run, labeled curve 1, included an average evaporation rate for each month, the monthly variations in demand tabulated in Table 10, and an elevation versus storage capacity relationship reflecting 50 years of sediment accumulation. The base run data was adopted for all the other simulation and reliability studies. Curve 2 is identical to the base run except a constant rather than monthly varying demand was used. Curve 3 is identical to the base run except the elevation versus storage capacity relationship which existed immediately after impoundment prior to sediment accumulation was used. Curve 4 is identical to the base run except net monthly evaporation rates were used which consisted of average evaporation minus average precipitation for each month. The base run evaporation did not include rainfall data. The reason that rainfall is typically included in net evaporation is to reflect the rainfall that reaches the reservoir that would have been abstracted prior to reaching the stream before the reservoir was built. Curve 5 was taken from the Fort Worth District reallocation report (USACOE, 1982) for purposes of comparison.

The sensitivity analysis showed that evaporation plays an extremely important role in determining the firm yield for a given storage capacity. Evaporation is a significant proportion of the total quantity of water withdrawn or released from the reservoir. Sedimentation significantly changes the firm yield for a given top of conservation pool elevation. The within year variation in total annual demand had relatively little effect on firm yield.

Reservoir Reliability. Reliability was expressed in this study as the percentage of the years simulated in which targeted monthly demands are met each month of the year. A failure consists of not meeting demands during one or more months of a year. The risk of failure is the percentage of the years simulated in which targeted demands are not met during one or more months. Thus, firm yield is by definition the yield provided with 100 percent reliability using the period of record reservoir inflows for the simulation. The



Simulation for top of conservation pool elevation of 455 feet and draft of 81 cfs (52.3 mgd).

Figure 8

Reservoir Simulation

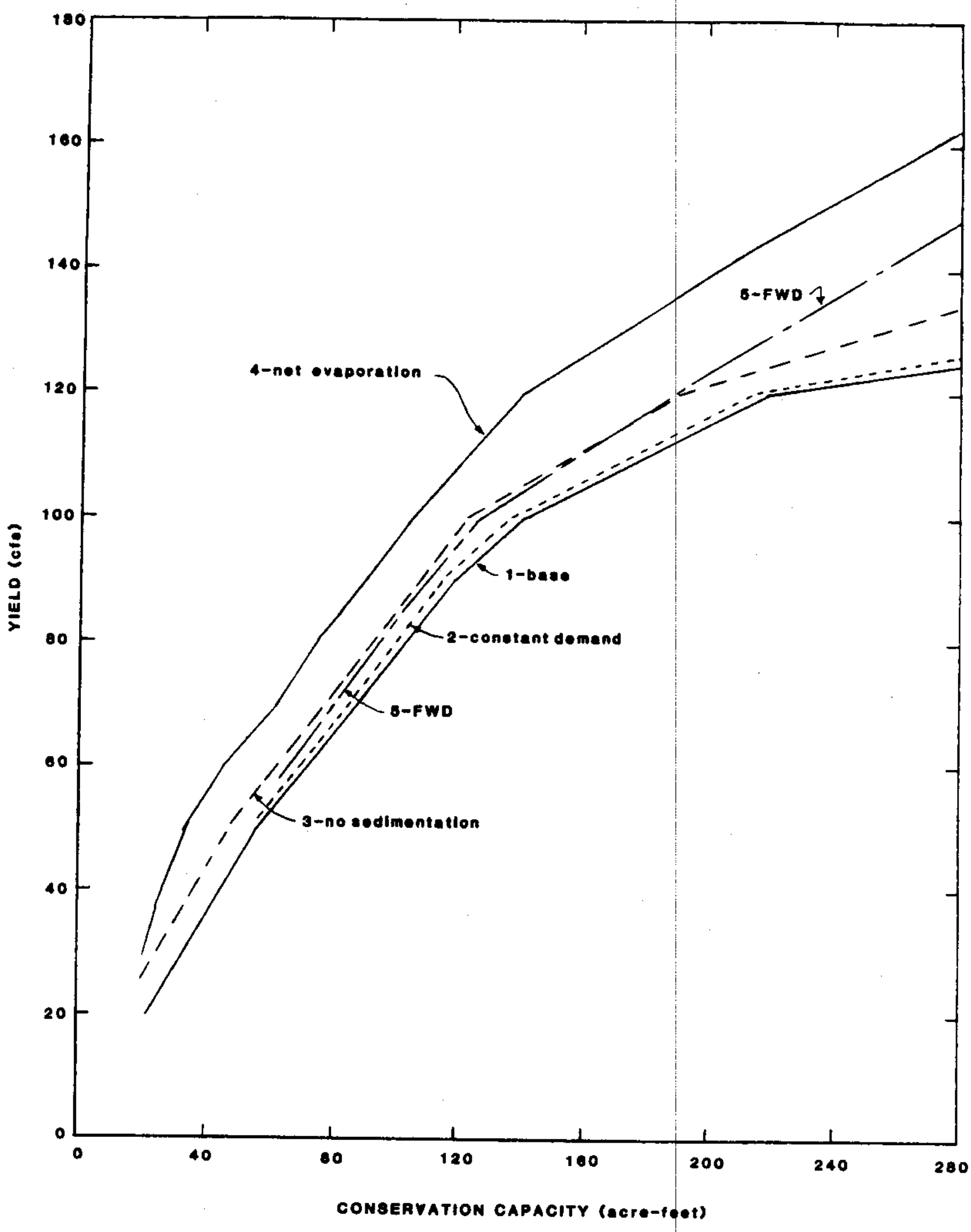


Figure 9
Storage Capacity Versus Firm Yield

MIT Simulation Model was used to compute reservoir reliability for alternative demand levels and conservation capacities.

As previously discussed, the period of record of reservoir inflows consisted of 76 years of average monthly streamflows. HEC-4 was used with this data to generate a 999 year synthetic sequence of monthly inflows. Reservoir reliablities for several alternative storage capacities and demands are tabulated in Tables 11 and 12 and plotted in Figure 10.

Estimation of Economic Losses Due to Water Shortages

The economic costs which will be incurred as a result of various levels of water shortage were estimated. A relationship between water shortage, expressed as a percentage of demand, and economic losses, in dollars, was compiled for various levels of water demand and provided as input data to the MIT Simulation Model to compute average annual losses.

Shortage Versus Loss Relationship. The estimation of economic losses due to water shortages involved: formulation of emergency demand management and supply augmentation plans; estimation of effectiveness and implementation costs for the emergency measures; and estimation of other losses to water users. Development of the shortage versus loss function is discussed in detail by the dissertation by Cabezas and is based largely upon information obtained from the literature regarding experiences in responding to water shortages in other areas and an analysis of water use practices in the study area. The following six emergency water management measures were selected as being representative of actions which could be taken in the event of a water shortage in the study area:

1) supply augmentation by relying on groundwater,
2) modification of the water price structure,
3) implementation of voluntary demand reduction programs,
4) implementation of mandatory demand reduction programs,
5) and importation of water by trucks or emergency pipelines.

The range of shortage severity of which each measure would be most applicable; effectiveness in reducing demands or agumenting supplies; and implementation costs were estimated. Other losses to residential and industrial water users and the municipality were also estimated. Shortage versus loss functions developed for various levels of water demand are presented in Table 14. A demand reduction of up to about 4.5 percent was estimated to result in essentially no economic losses. Past that level, the cost per 1000 gallons of

Table 11
Reservoir Reliability for Historical Inflows

Top of Conservation Pool Elevation (feet msl)	43.3	51.4	58.0	64.8	79.4
	Demands in MGD				
	Reliability in Percent				
437	61	53	47	43	28
447	97	93	89	84	68
451	99	97	95	92	80
455	100	100	98	96	88
462	100	100	100	100	95
470	100	100	100	100	98

Table 12
Reservoir Reliability for Synthetic Inflows

Top of Conservation Pool Elevation (feet msl)	43.3	51.4	58.0	64.8	79.4
	Demands in MGD				
	Reliability in Percent				
437	67	56	47	40	29
447	99	98	95	92	79
451	99	99	97	96	89
455	100	99	99	97	94
462	100	100	99	98	97
470	100	100	100	100	99

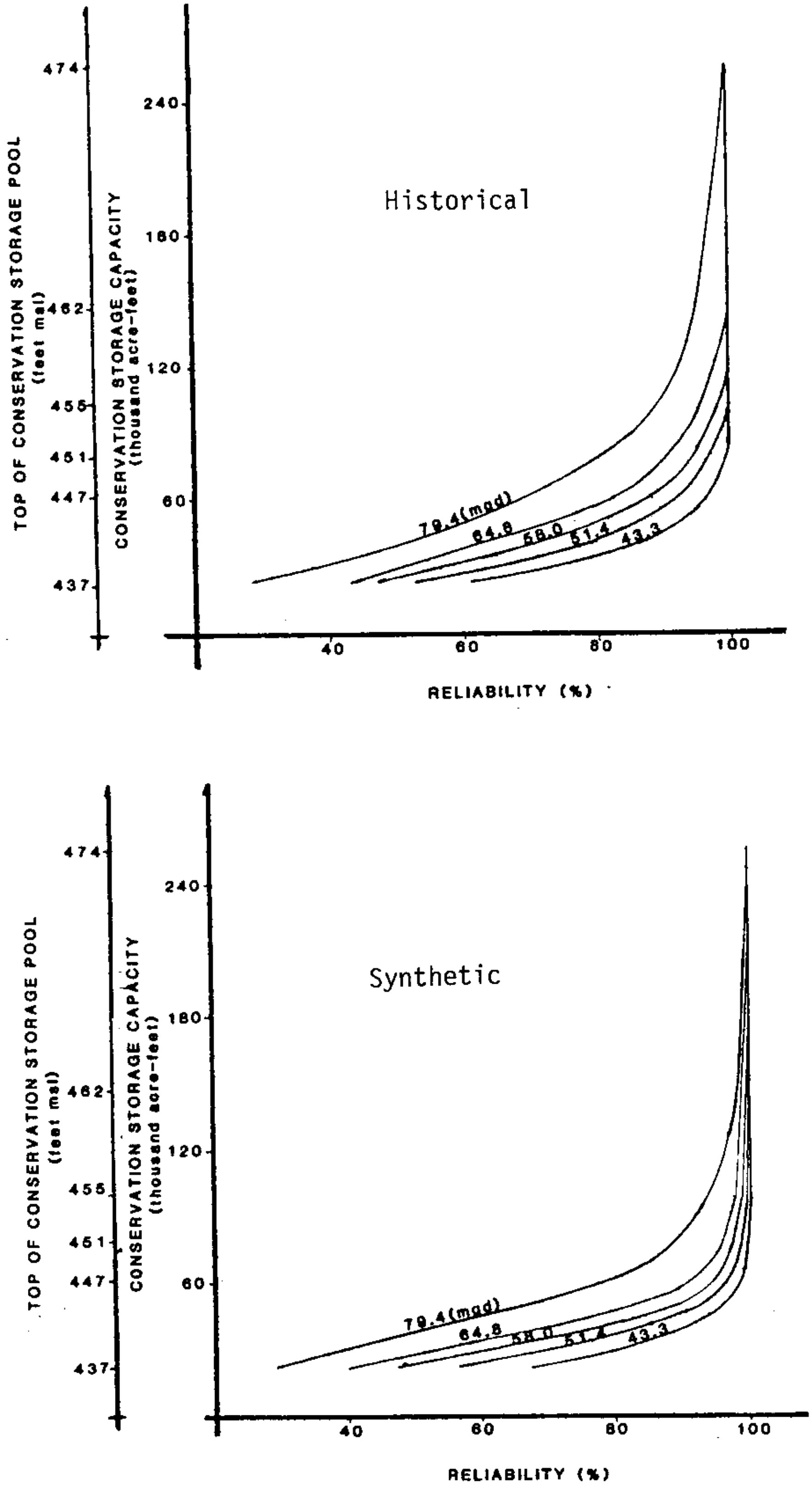


Figure 10
Reservoir Reliability

Table 13
Conservation Capacity Required for 100-Percent Reliability

100 Percent Reliability Yield in MGD	Top of Conservation Pool in Feet	
	Historical	Synthetic
43.3	452	458
51.4	455	464
58.0	457	467
64.8	461	470
79.4	474	---

Table 14
Water Shortage Versus Economic Loss Function

Management Measure	Unit Cost $/1000 gal	Accumulated Effectiveness %	Total Annual Cost $1000
		Water demand = 43.3 mgd	
Pricing	0	4.5	0
Voluntary reductions	0.96	13.5	1,524
Groundwater augmentation	1.45	25.5	4,170
Mandatory reductions	6.05	48.0	37,044
Importation	47.16	100.0	424,621
		Water demand = 51.4 mgd	
Pricing	0	4.5	0
Voluntary reductions	0.96	13.5	1,794
Groundwater augmentation	4.36	23.5	9,751
Mandatory reductions	5.17	48.0	44,783
Importation	47.16	100.0	504,863
		Water demand = 58.0 mgd	
Pricing	0	4.5	0
Voluntary reductions	0.95	13.5	2,018
Mandatory reductions	5.02	37.0	39,354
Groundwater augmentation	7.27	46.0	52,622
Importation	47.16	100.0	591,746
		Water demand = 64.8 mgd	
Pricing	0	4.5	0
Voluntary reductions	0.95	13.5	2,249
Mandatory reductions	4.80	37.0	42,038
Groundwater augmentation	10.18	46.0	60,617
Importation	47.16	100.0	674,103
		Water demand = 79.4 mgd	
Pricing	0	4.5	0
Voluntary reductions	0.95	13.5	2,745
Mandatory reductions	4.84	34.0	47,636
Groundwater augmentation	16.00	40.0	76,836
Importation	47.16	100.0	896,882

demand reduction or supply augmentation is a rapidly increasing nonlinear function of the severity of the water shortage.

Average Annual Losses. The average annual losses were computed along with reservoir reliability using the MIT Simulation Model with the relationships shown in Table 14 provided as input data. For each year of a simulation, the shortages occurring in each month are summed by the computer model to obtain the total yearly shortage. This shortage expressed as a percentage of demand combined with the shortage versus loss function results in a loss for the year. The average annual loss is the summation of the losses for all the years divided by the number of years in the simulation. Alternative storage capacities and demand levels were simulated using alternatively the 76-year historical record and 999-year synthetic sequence of monthly inflows. The results are tabulated in Tables 15 and 16 and shown graphically in Figure 11.

Implementation Costs

The reallocation study report prepared by the Fort Worth District includes a cost estimate for raising the top of conservation pool from 455 feet to 462 feet. Total costs in the estimated amount of $3,274,000, at 1982 price levels, would be incurred primarily in conjunction with relocating or reconstructing recreation facilities. Using an 83-year period of analysis (100 years minus 17 years of operation) and the 1982 discount rate of 7-7/8 percent, the equivalent annual costs would be $258,000. Cost estimates for implementation of storage reallocations were not developed in the present investigation.

Comparison of Analysis Results for Alternative Storage Reallocation Plans

Average annual losses for alternative water demand levels and top of conservation pool elevations are summarized in Table 17. The water supply losses are presented based alternatively on historical period of record and the 999-year sequence of synthetically generated streamflow data. The average annual losses are tabulated in Table 18 in terms of changes from losses associated with the present top of conservation pool elevation of 455 feet. The average annual losses in Table 18 are computed from the data in Table 17 by subtracting the losses associated with the 455-feet top of conservation pool elevation from the losses associated with the other alternative elevations. The data in Table 18 are plotted in Figures 12 through 16.

Table 15
Costs Due to Water Shortages
Historial Inflows

Top of Conservation Pool Elevation (feet msl)	Demands in MGD				
	43.3	51.4	58.0	64.8	79.4
	Average Annual Losses in $1,000				
447	23	297	1,198	3,170	12,473
451	0	41	377	958	6,737
455	0	0	12	186	2,006
462	0	0	0	0	398
470	0	0	0	0	0

Table 16
Costs Due to Water Shortages
Synthetic Inflows

Top of Conservation Pool Elevation (feet msl)	Demands in MGD				
	4.3	51.4	58.0	64.8	79.4
	Average Annual Losses in $1,000				
447	68	272	920	1,845	7,169
451	22	114	403	969	4,373
455	3	62	62	503	2,411
462	0	2	33	114	1,174
470	0	0	0	0	192

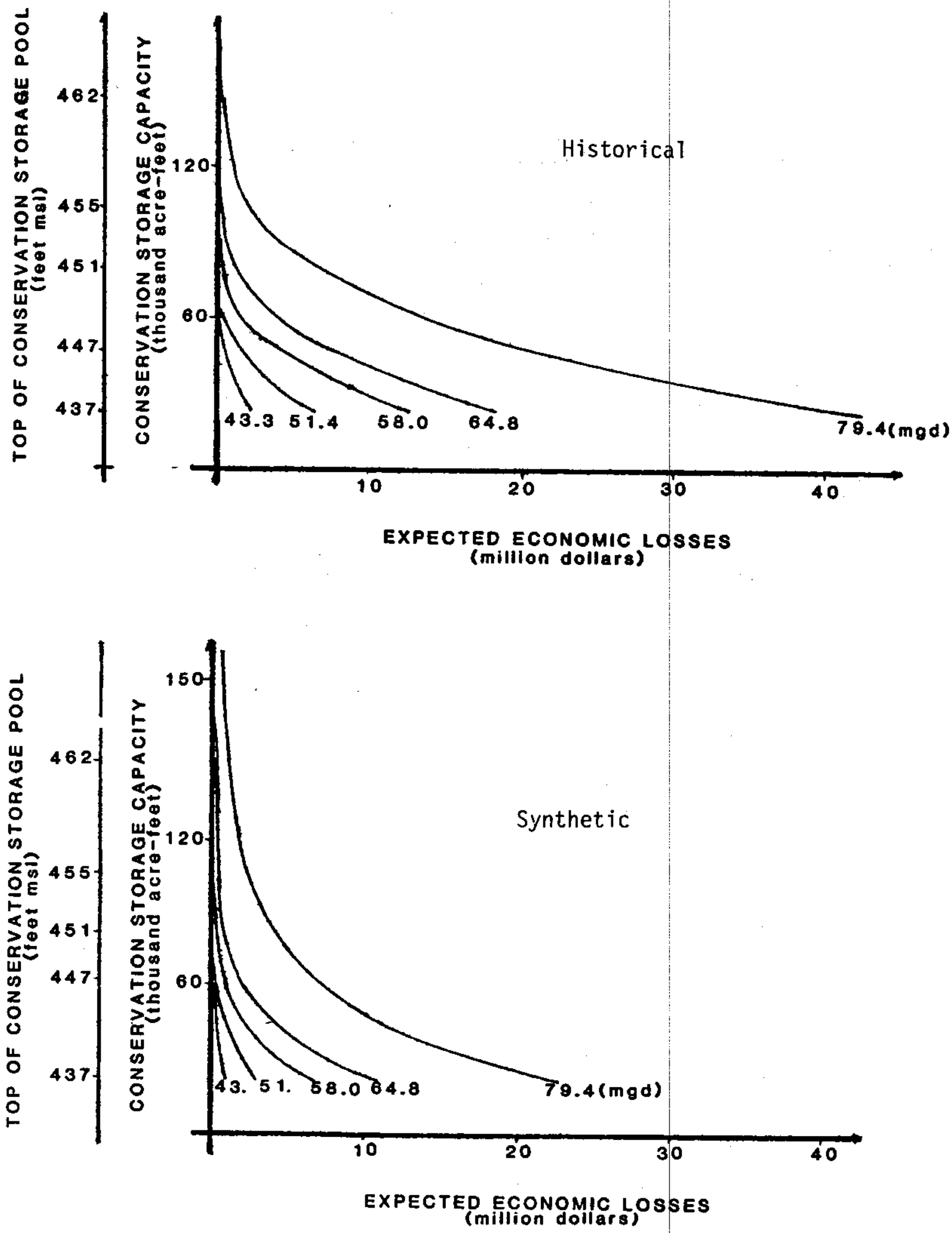


Figure 11

Costs Due to Water Shortages

Table 17
Average Annual Losses

Top of Conservation (feet msl)	Average Annual Losses (1000$)				
	Flood Control	Historical		Synthetic	
		Water Supply	Total	Water Supply	Total
		Demand = 43.3 mgd			
447	4,219	23	4,242	68	4,223
451	4,266	0	4,266	22	4,232
455	4,328	0	4,328	3	4,331
462	4,391	0	4,391	0	4,391
470	4,598	0	4,598	0	4,598
		Demand = 51.4 mgd			
447	4,219	297	4,516	272	4,491
451	4,266	41	4,307	132	4,398
455	4,328	0	4,328	62	4,390
462	4,391	0	4,391	2	4,393
470	4,598	0	4,598	0	4,598
		Demand = 58.0 mgd			
447	4,219	1,198	5,417	920	5,139
451	4,266	377	4,643	403	4,669
455	4,328	12	4,340	162	4,490
462	4,391	0	4,391	33	4,424
470	4,598	0	4,598	0	4,598
		Demand = 64.8 mgd			
447	4,219	3,170	7,389	1,845	6,064
451	4,266	958	5,224	969	5,235
455	4,328	186	4,514	503	4,831
462	4,391	0	4,391	114	4,505
470	4,598	0	4,598	0	4,598
		Demand = 79.4 mgd			
447	4,219	12,473	16,692	7,169	11,388
451	4,266	6,737	11,003	4,373	12,806
455	4,328	2,006	6,334	2,411	6,739
462	4,391	398	4,789	1,174	5,565
470	4,598	0	4,549	192	4,790

Table 18
Change in Average Annual Losses
(From the Present 455-feet Top of Conservation Pool)

Top of Conservation (feet msl)	Change in Average Annual Losses (1000$)				
		Historical		Synthetic	
	Flood Control	Water Supply	Total	Water Supply	Total
		Demand = 43.3 mgd			
447	-109	23	- 86	65	- 44
451	- 62	0	- 62	19	- 43
455	0	0	0	0	0
462	63	0	63	-3	60
470	270	0	270	-3	267
		Demand = 51.4 mgd			
447	-109	297	188	210	101
451	- 62	41	-21	70	8
455	0	0	0	0	0
462	63	0	63	-60	3
470	270	0	270	-62	208
		Demand = 58.0 mgd			
447	-109	1,186	1,077	758	649
451	- 62	365	303	241	179
455	0	0	0	0	0
462	63	-12	51	-129	-66
470	270	-12	258	-162	108
		Demand = 64.8 mgd			
447	-109	2,984	2,875	1342	1,233
451	- 62	772	710	466	404
455	0	0	0	0	0
462	63	-186	-123	-389	-326
470	270	-186	- 84	-503	-233
		Demand = 79.4 mgd			
447	-109	10,467	10,358	4,758	4,649
451	- 62	4,731	4,669	1,962	1,900
455	0	0	0	0	0
462	63	-1,608	-1,545	-1,237	-1,174
470	270	-2,006	-1,736	-2,219	-1,949

Table 19
Comparison of Storage Reallocations

Top of Conservation Pool (feet)	447	451	455	462	470
Water Supply Capacity (acre-feet)	62,000	80,000	104,100	147,500	220,000
Flood Control Capacity (acre-feet)	595,400	577,400	553,300	509,900	437,400
Firm Yield (mgd)	36.3	43.3	52.4	64.6	77.6
Design Flood Return Interval (years)	113	111	109	84	57
Reliability (percent)					
Demand = 43.3 mgd (1990)	99	99	100	100	100
Demand = 51.4 mgd (2000)	98	99	99	100	100
Demand = 58.0 mgd (2010)	95	97	99	99	100
Demand = 64.8 mgd (2020)	92	96	97	98	100
Demand = 79.4 mgd (2040)	79	89	94	97	99
Change in Average Annual Losses ($1,000)					
Demand = 43.3 mgd (1990)	-44	-43	0	60	267
Demand = 51.4 mgd (2000)	101	8	0	3	208
Demand = 58.0 mgd (2010)	649	179	0	-66	108
Demand = 64.8 mgd (2020)	1,233	404	0	-326	-233
Demand = 79.4 mgd (2040)	4,649	1,900	0	-1,174	-1,949

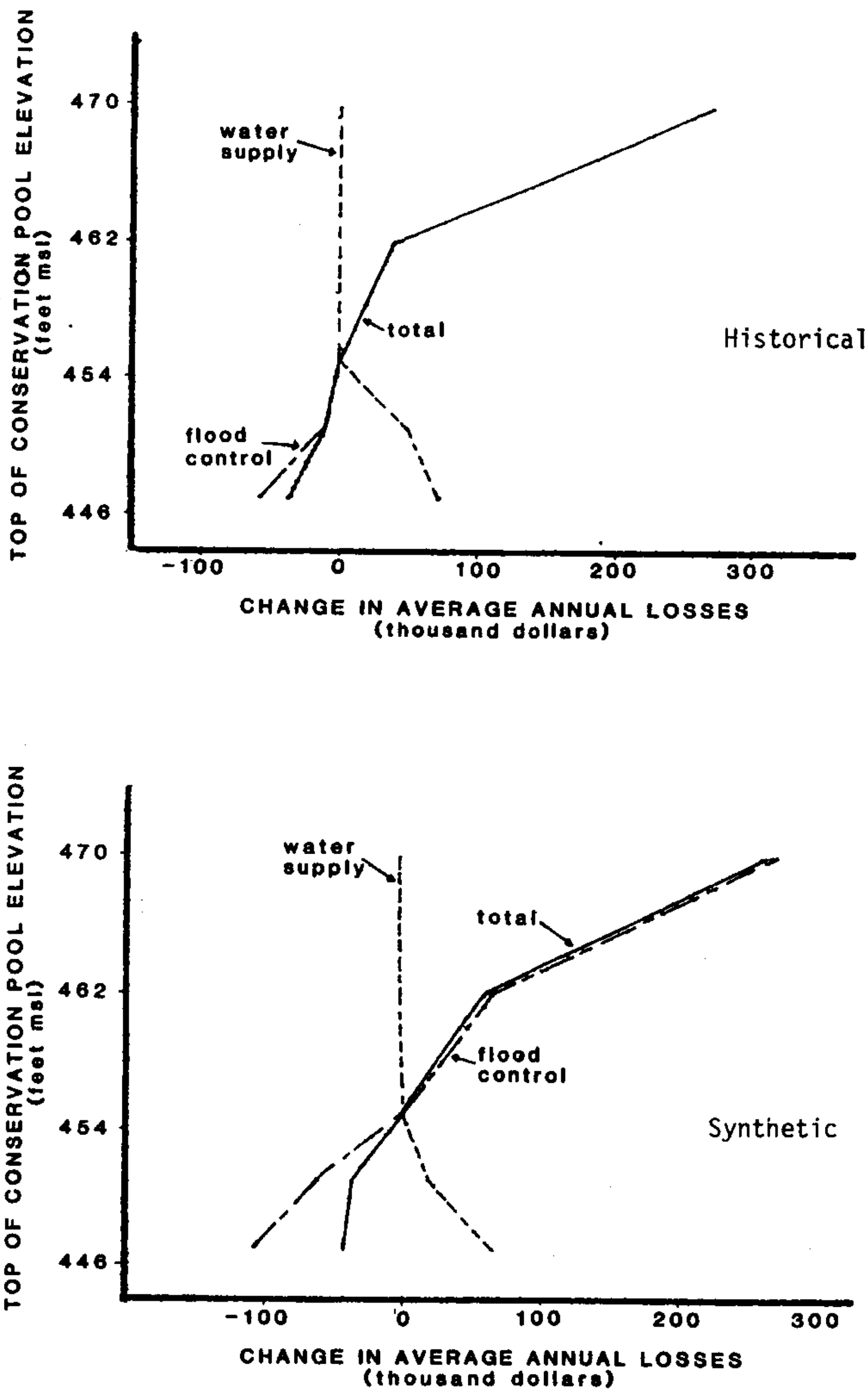


Figure 12

Comparison of Losses for 43.3 MGD Demand

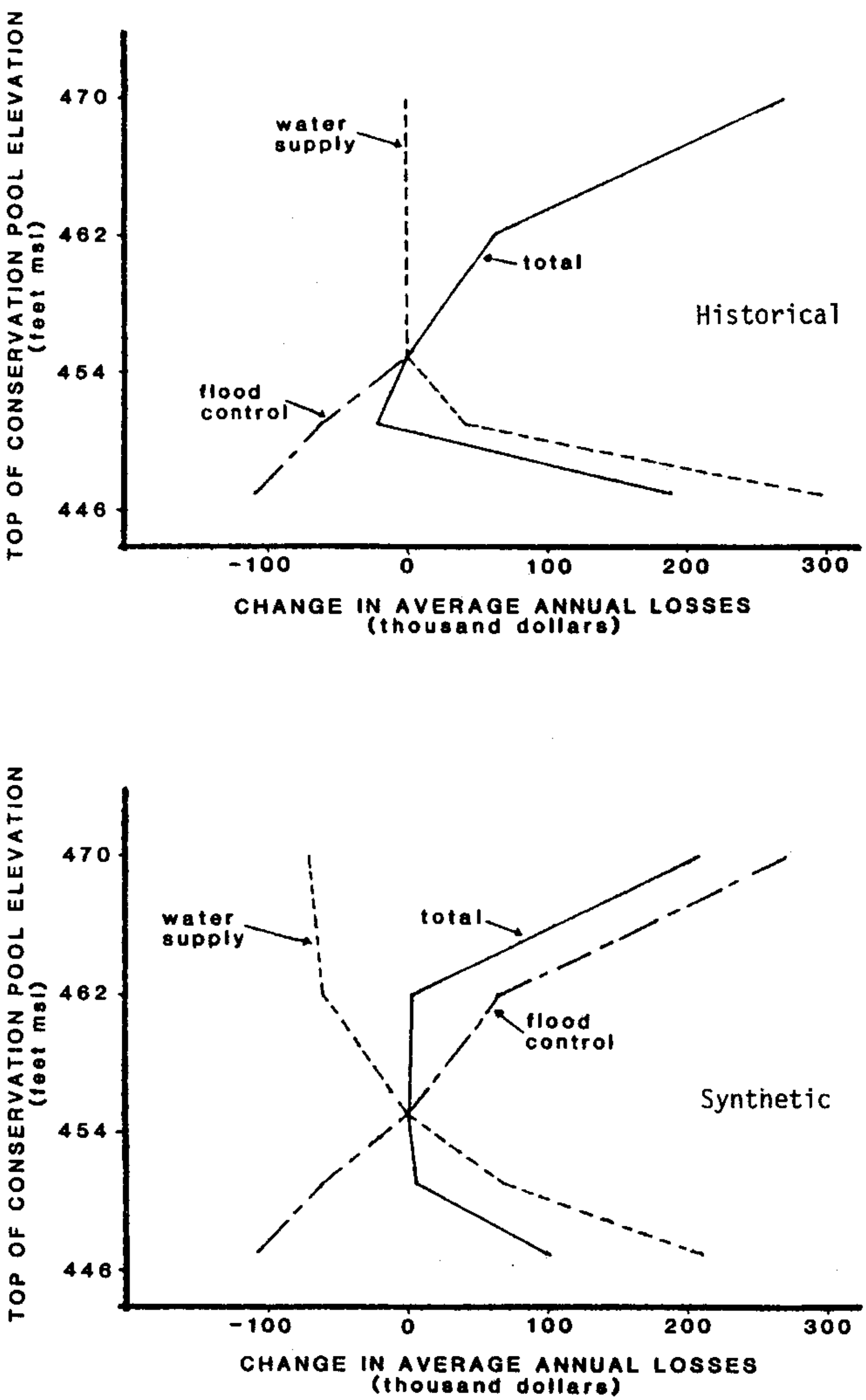
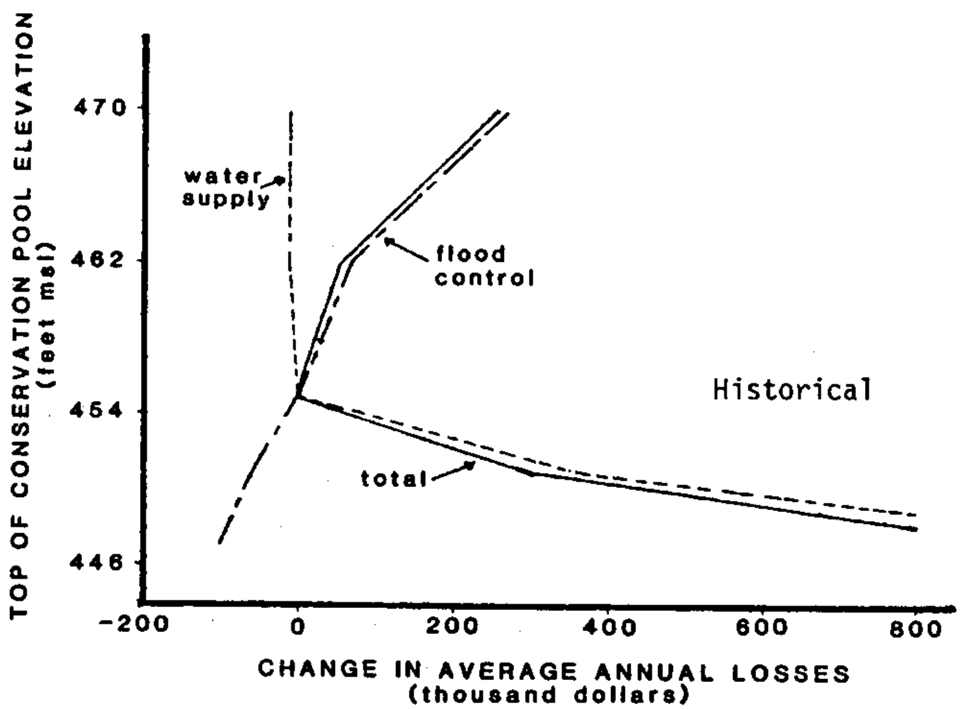


Figure 13

Comparison of Losses for 51.4 MGD Demand



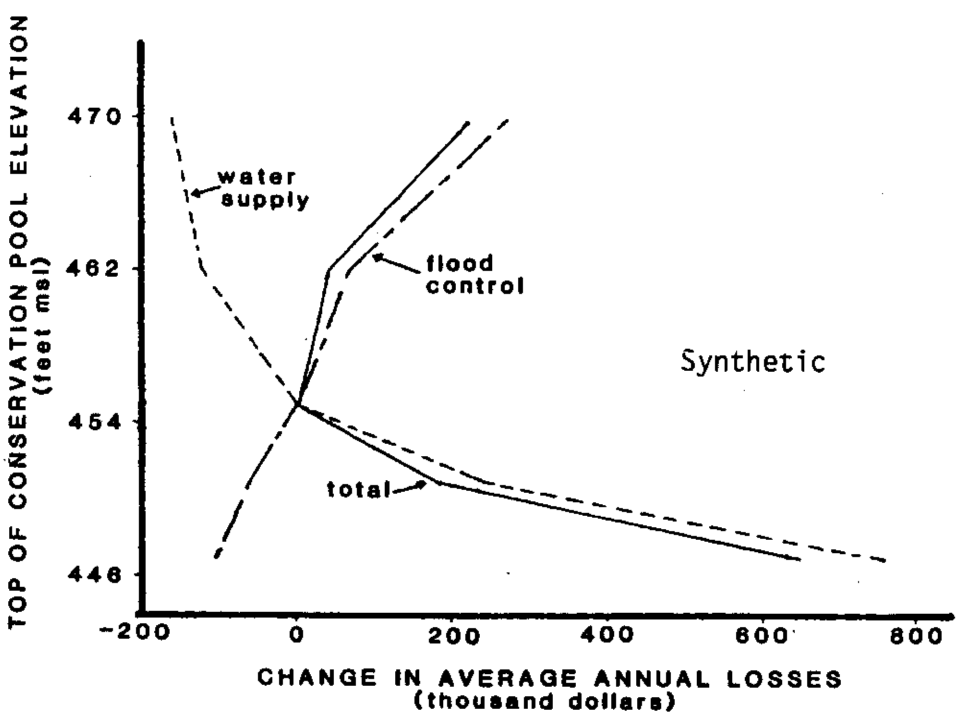
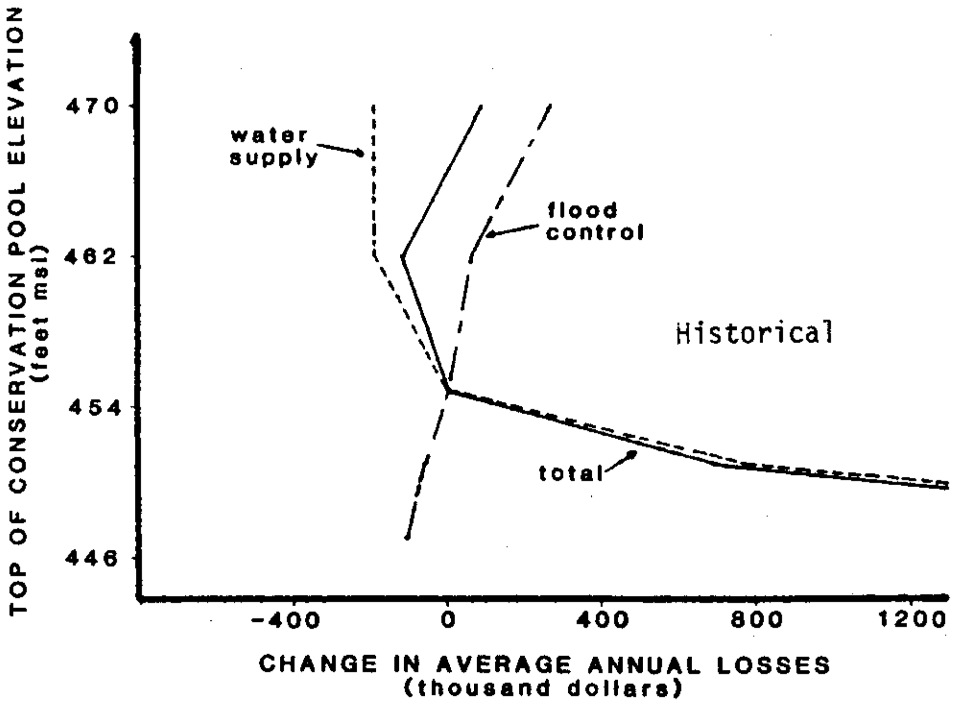


Figure 14

Comparison of Losses for 58.0 MGD Demand



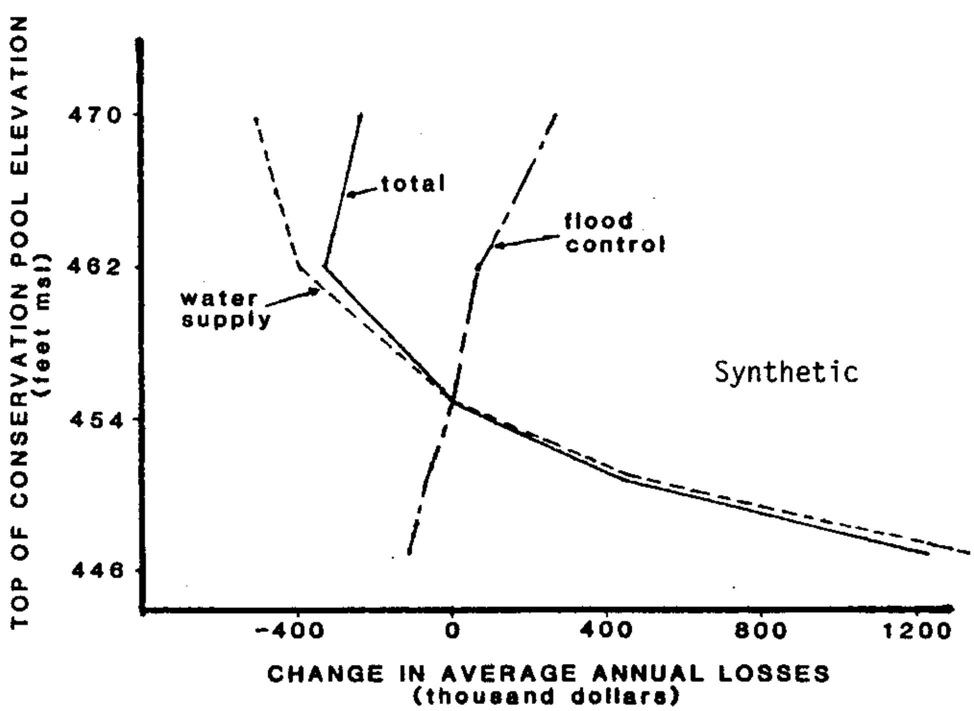
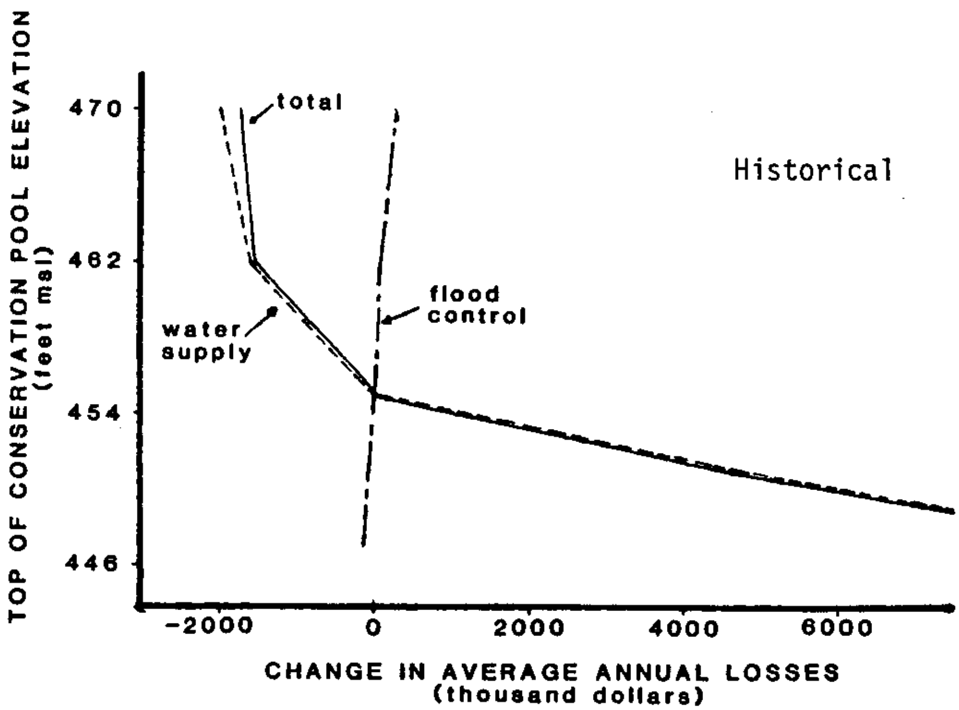


Figure 15

Comparison of Losses for 64.8 MGD Demand



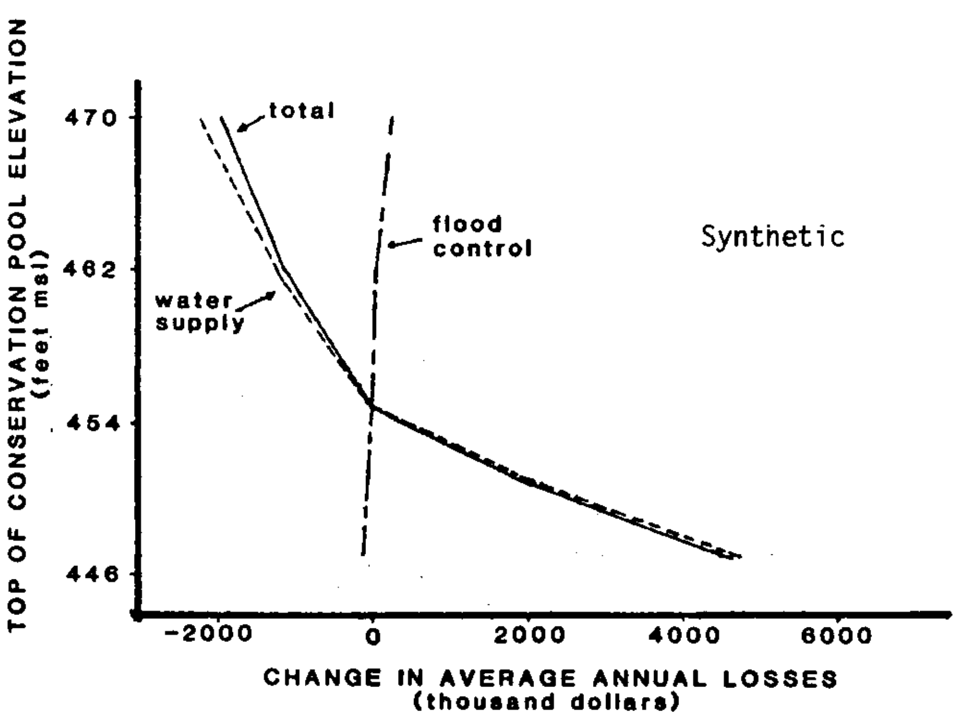


Figure 16

Comparison of Losses for 79.4 MGD Demand

Selected analysis results are summarized in Table 19. The summary includes the conservation and flood control storage capacities used in all the analyses, which reflect 50-years (2015 conditions) of sediment accumulation. Firm yields are from Figure 9, computed with HEC-5 based on historical period of record streamflows. The design flood return intervals are reproduced from Table 6. The reservoir reliabilities and total average annual losses based on the synthetic streamflow data were selected for inclusion in Table 19. Reliabilities and average annual losses are a function of water demands. The years at which the indicated demands are projected to occur are shown in parenthesis in the table.

A firm yield of 52.4 mgd was computed for the present top of conservation pool elevation of 455 feet. The Corps of Engineers estimated a firm yield of 54.9 mgd. The Corps of Engineers demand projections, which were adopted in the present study, indicate that water demands will increase to 51.4 mgd by the year 2000. Thus, the firm yield exceeds projected demands up to the year 2000. Demands of 43.3, 51.4, 58.0, 64.8, and 79.4 mgd are projected to occur in the years 1990, 2000, 2010, 2020, and 2040, respectively, based on the assumption that long-term demand management strategies are not implemented. The alternative reduced set of demands, developed by the Corps of Engineers based on the assumption of long-term demand management, are not addressed here. The present economic evaluation is also limited to alternative top of conservation pool elevations of 447, 451, 455, 462, and 470 feet. However, the computations could be easily repeated for other alternative operating policies and water demands as well.

The 1990 water demand of 43.3 mgd is well under the firm yield of 52.4 mgd provided by the present 455-feet top of conservation pool elevation. The reliability of the existing project is estimated to be 100 percent (greater than 99.5 percent) based on either historical or synthetic streamflow data. With the top of conservation pool lowered to elevation 447 feet, the increase in water supply losses is less than the decrease in flood losses. Based strictly on minimizing the sum of estimated water supply and flood losses, the reservoir should presently be operated at a top of conservation pool elevation in the range of 447 to 451 feet. However, implementation costs and losses in recreation benefits would likely render such a reallocation infeasible.

The results of the economic evaluation for the year 2000 projected water demand of 51.4 mgd are presented in Figure 13. Using the synthetic streamflow data, raising the top of conservation pool from 455 feet to 462 feet results in an estimated increase in flood losses about equal to the decrease in water supply losses. Likewise, lowering the top of conservation pool elevation to 451 feet results in an approximate balance between the increase in water supply losses and decrease in flood losses. Thus, the total losses are about the same for top of conservation pool elevations of 451, 455, or 462. Using the historical streamflow data results in a minimum total average annual loss occurring at a top of conservation elevation of 451 feet. The 451-feet total average losses are $21,000 less than those for the 455-feet operating policy, which would again be expected to be less than implementation costs and loss recreation benefits associated with a storage reallocation. Using either historical or synthetic streamflow data, the increase in flood losses significantly outweighs the decrease in water supply losses for 470 feet, and water supply losses outweigh flood control for 447 feet. Thus, the two extreme top of conservation pool elevations of 447 and 470 feet are indicated to be economically infeasible. The 51.4 mgd demand is still less than the 52.4 mgd firm yield provided by the present top of conservation pool elevation of 455 feet. The reliability is estimated to be 100 percent based on the historical streamflow data and 99 percent based on the synthetic data. Thus, a demand of 51.4 mgd is not large enough to justify a storage reallocation.

The results of the economic evaluation for the year 2010 projected demand of 58.0 mgd are presented in Figure 14. The total average annual losses are a minimum for a top of conservation pool elevation of 455 feet. Increasing the conservation capacity results in increases in flood control losses greater than the corresponding decreases in water supply losses. Likewise, increasing the flood control capacity results in increases in water supply losses greater than the corresponding decreases in flood control losses. Thus, the economically optimum storage allocation has a firm yield less than the demand and a reliability slightly less than 100 percent.

The results of the economic evaluation for the year 2020 projected demand of 64.8 mgd are plotted in Figure 15. Top of conservation elevation 462 feet is the economically optimum alternative capacity allocation, assuming the discounted costs for implementing the reallocation are less than the decrease in average annual losses. Raising the top of conservation pool to elevation 462 feet is estimated to reduce total average annual losses $123,000 or $389,000, respectively, based on historical or synthetic streamflow data. A top of conservation pool elevation of 462 feet would provide a firm yield of

64.6 mgd which is essentially equal to the 64.8 mgd demand. The reliability would be 96 or 97 percent. The flood control capacity would be reduced from an estimated 109-year to 84-year return interval design flood.

The year 2040 projected water demand of 79.4 mgd would justify raising the top of conservation pool to elevation 470 feet or possibly higher. Raising the pool level from 455 feet to 470 feet reduces the estimated annual losses by $1,736,000 or $1,944,000 based on historical or synthetic streamflow data, respectively. At a demand of 79.4 mgd, the decrease in water supply losses achieved by raising the pool level greatly exceeds the corresponding increase in flood control losses.

In conclusion, the analysis results indicate that a demand of 64.8 mgd would result in raising the top of conservation pool to elevation 462 feet being economically feasible. The 64.8 mgd demand level is projected to occur about the year 2020. The 2040 demand of 79.4 mgd would economically justify raising the pool to elevation 470 feet.

The average annual water supply losses are computed based on the assumption that the storage reallocation is implemented before the beginning of a critical drought period. A drought will typically last several years. For example, the previously discussed simulation indicated that a full reservoir in May 1953 was emptied by February 1955 and was full again in March 1957. Consequently, to achieve the expected annual reduction in water supply losses, the storage reallocation should be implemented several years before actually reaching the indicated demand.

CHAPTER 7

SEASONAL RULE CURVE OPERATION

Seasonal rule curve operation can be adopted to reflect seasonally varying conditions. A rule curve specifies the top of conservation pool elevation as a function of time of the year. Storage capacity is reallocated between flood control and conservation purposes in a set annual cycle. The feasibility of seasonal rule curve operation in Texas is addressed in detail in the thesis by Tibbets and briefly discussed in this chapter.

Seasonally Varying Factors Affecting Reservoir Operation

Risk of flooding, flood damage susceptibility, water supply demands, and water availability vary seasonally. Unlike many parts of the world in which almost all floods occur in a distinct season of the year, floods can occur at any time in Texas. However, the likelihood of flooding is significantly higher during certain months than in others. Seasonal variations in flood damage susceptibility are related primarily to agricultural activities. The extent of damage depends upon whether the flood occurs during the growing season. The majority of the flood control benefits attributed to reservoirs in Texas are related to agriculture. Municipal as well as agricultural water demands are highly seasonal. The seasonality of municipal demands is due largely to summer lawn watering. Hydroelectric power demands are also higher during the summer due to air conditioning. Most reservoir recreation occurs during the summer. Reservoir evaporation is much higher during the summer. The availability of streamflow for water supply purposes is relatively low during the summer when the demands are highest.

Seasonally varying factors affecting reservoir operation are generally related to weather. The climate of Texas is characterized by variations in the weather, both geographically and temporally. Variations in precipitation and temperature are determined primarily by the confluence of warm, moist Gulf air and relatively cool, dry air from the continental United States. The western half of the state has a semi-arid, continental-type climate, characterized by rapid and drastic fluctuations in temperature. The remainder of the state is influenced by a humid, subtropical climate, having moderate temperatures. Although some snow does occur, most precipitation is in the form of rain. Generally, precipitation decreases from east to west across the state at a rate of about one inch every 15 miles. Average annual precipitation ranges from more than 56 inches at the eastern border to less than eight inches in the most western part of the state. Statewide average, minimum, and maximum monthly precipitation during the period 1931 through 1979 are tabulated in Table 20. From a statewide perspective, May is the wettest month. Winter is the driest time of the year.

Patterns of seasonal precipitation vary considerably for different areas of the state (Orton, 1969). Rains generally occur most frequently in late spring as a result of squall-line thunderstorms. Most areas, including most of the High and Low Rolling Plains, Edwards Plateau, North Central, and South Central Texas, show a peak in May. Rainfall in the Pecos Valley, most of southern Texas, the lower Rio Grande Valley, and the coastal region, peaks in September with a secondary peak in May. On the High Plains, particularly the northern portion, a significant percentage of the total annual precipitation occurs during the summer months, following the May peak. Throughout the central part of the state, July and August are relatively dry months. In the mountainous Trans-Pecos area of West Texas, afternoon thundershowers during July, August, and September account for most of the annual rainfall. Throughout most of East Texas, rainfall is fairly evenly distributed throughout the year.

Tropical cyclones, particularly tropical storms and hurricanes, are a perennial threat to the Texas Gulf coastal region during the summer and autumn. Essentially all the tropical cyclones that affect the Texas coast originate in the Gulf of Mexico, Caribbean Sea or in other parts of the North Atlantic Ocean. Although the hurricane season extends from June to October, tropical cyclones are most frequent in August and September and rarely affect the coast before mid-July or after mid-October. Hurricanes contribute large quantities of precipitation in addition to producing high winds and storm tides.

Griffiths and Ainsworth (1981) describe Texas weather, including major flooding events, during the period 1880 through 1979. The Texas Almanac and Industrial Guide (A.H. Belo Corp., 1984) contains a list of exceptionally destructive storms in the state since 1766. Forty-two of the most severe floods were distributed among months of the year as follows: January, 1 flood; February, 1; March, 2; April, 5; May, 7; June, 6; July, 4; August, 2; September, 10; October, 1; November, 1; and December 2 floods. Thus, almost a fourth of the extreme flood events occurred in September and a third occurred in March, April, and May.

Table 20
Monthly Statewide Precipitation During 1931-1979

Month	Statewide Precipitation (inches)		
	Average	Minimum	Maximum
January	1.7	0.2	3.9
February	1.7	0.3	2.9
March	1.6	0.3	3.2
April	2.5	0.8	6.7
May	3.4	1.2	7.1
June	2.8	0.7	5.6
July	2.4	0.9	5.1
August	2.4	0.6	5.7
September	3.2	0.6	6.9
October	2.4	0.0	5.9
November	1.7	0.1	5.3
December	1.8	0.2	4.0

Source: Griffiths and Ainsworth (1981)

Table 21 is a tabulation of recorded precipitation events in which a station received 15 inches or more durng a 24-hour period (Griffiths and Ainsworth, 1981). Forty-four percent of these extreme rainfall measurements occurred in the month of September. Most of the other events occurred during the summer months. The data in Table 21 are based on official precipitation gage readings. Unofficial measurements of 45 inches of rainfall was reported northwest of Alvin during Tropical Storm Claudette in July 1979 along with several other reports of more than 25 inches near the cities of Freeport and Clear Lake. During a storm in September 1921, more than 38 inches of rain was unofficially reported to have fallen in 24 hours at a point north of Thrall, in Central Texas.

Agricultural and municipal water demands, hydroelectric power demands, recreational use, and evaporation are highest during the hot summer months concurrently with relatively low reservoir inflows. Consequently, raising the top of conservation pool elevation in April to capture additional Spring rainfall for use during the summer should enhance conservation operations. However, extreme flood events tend to occur during the period from April through October. September is a particularly critical month for flooding. Consequently, the time periods during which flood control capacity and water supply capacity are most needed significantly overlap.

Seasonal Rule Curve Operation in Texas

Although seasonal rule curves are fairly common in many parts of the United States, this type of operating policy has not been widely adopted in Texas. The top of conservation pool has been varied seasonally at four reservoirs in the state.

Two Corps of Engineers projects, Lake O' the Pines and Wright Patman in the northeast corner of the state, are operated in accordance with seasonal rule curves. The operating curve for Lake O' the Pines provides for raising the top of conservation pool 1.5 feet from mid-May through mid-September for recreation purposes. The rule curve for Wright Patman varies significantly during the year in response to an interim operating agreement with the conservation storage sponsor to provide additional municipal and industrial water supply. The top of conservation pool is constant from November through March and varies with date from April through October. The top of conservation pool peaks on June 1 at a level 6.9 feet above the winter pool

Table 21
Gaged Rainfall Events of 15 Inches or More During 24 Hours

Rainfall (inches)	Station	County	Date
29.05	Albany	Shackelford	4 Aug 1978
25.75	Alvin	Brazoria	26 Jul 1979
25.27	Orange	Orange	Sep 1963
25.24	Point Comfort	Calhoun	Jun 1960
25.06	Galveston Airport	Galveston	Sep 1958
25.06	Harleton	Harrison	Apr 1966
25.01	Armstrong	Kenedy	Sep 1967
23.11	Taylor	Williamson	9-10 Sep 1921
21.02	Kaffie Ranch	Jim Hogg	12 Sep 1971
20.70	Hye	Blanco	11 Sep 1952
20.60	Montell	Uvalde	27 Jun 1913
19.29	Danevang	Wharton	27-28 Aug 1945
19.20	Benavides No. 2	Duval	11 Sep 1971
19.03	Austin	Travis	9-10 Sep 1921
18.00	Fort Clark	Kinney	14-15 Jun 1899
17.76	Port Arthur	Jefferson	27-28 Jul 1943
17.47	Blanco	Blanco	11 Sep 1952
16.72	Freeport 2NW	Brazoria	26 Jul 1979
16.05	Smithville	Bastrop	30 Jun 1940
16.02	Hills Ranch	Travis	10 Sep 1921
16.02	Pandale	Val Verde	27 Jun 1954
16.00	Hempstead	Waller	24 Nov 1940
15.87	Anahuac	Chambers	27-28 Aug 1945
15.80	Orange	Orange	18 Sep 1963
15.71	Matagorda	Matagorda	1 May 1911
15.69	Whitsett 2SW	Live Oak	22 Sep 1967
15.65	Houston Airport	Harris	27-28 Aug 1945
15.60	Eagle Pass	Maverick	29 Jun 1936
15.49	Deweyville 5S	Orange	28 Oct 1970
15.20	World's End Ranch	Kerr	2 Aug 1978
15.00	Mercedes	Hidalgo	5 Sep 1933
	Oct 1949 - 8 other stations reported 20" or more		
	Sep 1967 - 17 other stations reported 20" or more		

Source: Griffiths and Ainsworth (1981)

level. A permanent reallocation of flood control to conservation is planned for Wright Patman Reservoir upon completion of construction of Cooper Reservoir upstream. The seasonal rule curve is being followed until that time.

The top of conservation pool elevations for Falcon and Amistad Reservoirs on the Rio Grande River can be, at the discretion of the International Boundary and Water Commission, temporarily raised for seasonal rule curve operation. However, the optional encroachment into the flood control pool does not necessarily occur routinely each year and the magnitude of encroachment can be varied within a fixed maximum limit.

Waco Reservoir Case Study

Monthly data descriptive of seasonal variations in factors pertinent to operation of Waco Reservoir are provided in Tables 22 through 25. Table 22 is a tabulation of monthly values of average rainfall in the watershed above the dam, streamflow at the project site, and pan evaporation, which is reproduced from the regulation manual (USACOE, 1971) and monthly water demands computed by multiplying the 1990 annual demand by the monthly percentages from Table 10. These data are expressed as a percentage of annual values in Table 23. Water availability peaks in May. Thirty-eight percent of the average annual streamflow into the reservoir occurs in April and May. The average inflow rate is a minimum in July and August. Municipal and industrial water supply demands are a maximum in July and August. Evaporation, which represents a very significant withdrawal of water from the reservoir, is also a maximum in July and August. Assuming the water surface at the top of conservation pool and a pan coefficient of 0.69, reservoir evaporation varies from 1,200 acre-feet in January to 5,300 acre-feet in July.

Dugas (1983) performed a statistical analysis of daily precipitation data from 36 stations located throughout the state. The probabilities of various amounts of precipitation being equalled or exceeded during each week of the year are shown in Table 24 for a precipitation station at Temple, which is located near Waco Reservoir.

Major floods on the watershed above Waco Reservoir are listed in Table 25. April, May, and September have been the dominant flooding months in the historical record. Floodplain activities are largely agricultural crops, and damage susceptibility is thus highly seasonal. The discharge versus damage curves indicate that damages for a flood during May through August are several

Table 22
Pertinent Monthly Data for Waco Reservoir

Month	Average Rainfall (inches)	Average Streamflow (acre-feet)	Average Pan Evaporation (inches)	1990 Water Demand (acre-feet)
January	2.26	18,350	2.76	3,250
February	2.39	24,540	3.63	2,750
March	2.09	26,620	6.09	3,170
April	3.83	47,900	7.31	3,330
May	4.83	73,570	7.58	3,920
June	2.88	30,390	10.26	4,570
July	2.14	12,650	12.60	5,690
August	1.67	8,670	11.49	5,780
September	3.00	18,250	7.84	4,920
October	2.58	21,970	6.71	4,200
November	2.19	16,320	4.26	3,480
December	2.50	19,930	3.09	3,520
Annual	32.36	319,160	83.62	48,500

Table 23
Pertinent Monthly Data for Waco Reservoir
As a Percentage of Annual Values

Month	Average Rainfall (%)	Average Streamflow (%)	Average Pan Evaporation (%)	Water Demand (%)
January	7.0	5.8	3.3	6.6
February	7.4	7.7	4.3	6.2
March	6.4	8.3	7.3	6.4
April	11.8	15.0	8.7	7.0
May	14.9	23.1	9.1	7.9
June	8.9	9.5	12.3	9.5
July	6.6	4.0	15.0	11.5
August	5.2	2.7	13.7	11.7
September	9.3	5.7	9.4	10.3
October	8.0	6.9	8.0	8.5
November	6.8	5.1	5.1	7.3
December	7.7	6.2	3.7	7.1
Annual	100.0	100.0	100.0	100.0

Table 24
Precipitation Amount Probabilities for 1-Week Period at Temple

PERIOD BEGINS	PRECIPITATION MEANS, MAXIMUMS, AND PROBABILITIES FOR A 1-WEEK PERIOD												
	MEAN		MAXIMUM		PROBABILITY (%) OF RECEIVING AT LEAST THE FOLLOWING AMOUNTS OF PRECIPITATION								
	MM	IN	MM	IN	MM 0.25 / IN 0.01	6.4 / 0.3	12.7 / 0.5	25.4 / 1.0	38.1 / 1.5	50.8 / 2.0	76.2 / 3.0	101.6 / 4.0	127.0 / 5.0
MAR 1	13	0.54	87	3.4	72	53	39	20	10	5	2	1	1
MAR 8	12	0.48	91	3.6	78	51	35	16	8	4	2	1	1
MAR 15	12	0.51	83	3.3	74	50	36	18	9	5	2	1	1
MAR 22	14	0.57	113	4.4	74	53	38	21	11	6	3	1	1
MAR 29	12	0.50	148	5.9	66	46	33	18	10	5	2	1	1
APR 5	19	0.75	93	3.7	77	58	45	27	17	11	4	2	1
APR 12	20	0.82	117	4.6	75	56	45	30	19	13	5	3	2
APR 19	28	1.13	168	6.6	85	69	58	40	28	19	10	5	3
APR 26	35	1.41	177	7.0	83	72	63	47	35	26	15	8	4
MAY 3	23	0.94	126	5.0	85	67	54	35	22	14	6	3	2
MAY 10	29	1.15	164	6.5	83	70	59	42	29	20	10	5	3
MAY 17	29	1.16	202	8.0	77	65	55	40	28	21	11	6	4
MAY 24	25	1.02	279	11.0	75	60	50	35	24	18	9	5	3
MAY 31	18	0.73	84	3.3	67	54	43	27	17	11	4	2	1
JUN 7	13	0.52	115	4.6	56	41	31	19	11	7	3	2	1
JUN 14	18	0.73	144	5.7	56	45	37	26	18	12	6	3	2
JUN 21	20	0.81	208	8.2	63	49	39	28	19	14	7	4	3
JUN 28	13	0.52	132	5.2	60	40	30	18	11	7	3	2	1
JUL 5	10	0.42	101	4.0	51	35	26	15	9	5	2	1	1
JUL 12	12	0.48	107	4.2	50	37	28	17	10	7	3	2	1
JUL 19	11	0.46	243	9.6	55	37	27	16	10	6	3	2	1
JUL 26	12	0.50	201	7.9	49	33	26	17	11	8	4	2	2
AUG 2	10	0.43	117	4.6	42	31	24	16	10	6	3	2	1
AUG 9	10	0.42	104	4.1	51	37	27	15	9	5	2	1	1
AUG 16	11	0.46	128	5.1	55	36	27	16	10	6	3	2	1
AUG 23	14	0.57	75	3.0	62	49	37	22	12	7	3	1	1
AUG 30	15	0.59	109	4.3	63	45	34	21	13	9	4	2	1
SEP 6	24	0.96	303	11.9	69	53	43	31	23	17	9	5	4
SEP 13	19	0.76	178	7.0	66	48	39	26	18	12	6	4	3
SEP 20	17	0.69	123	4.9	70	53	41	25	15	10	4	2	1
SEP 27	14	0.58	221	8.7	53	37	29	20	13	9	5	3	2
OCT 4	16	0.64	185	7.3	60	42	34	22	15	10	5	3	2
OCT 11	20	0.81	202	8.0	55	44	36	26	20	14	8	5	3
OCT 18	17	0.69	128	5.1	60	50	41	26	17	11	4	2	1
OCT 25	18	0.71	112	4.4	65	54	43	27	16	10	3	2	1
NOV 1	17	0.69	85	3.4	62	47	38	25	17	11	5	3	2
NOV 8	15	0.61	126	5.0	61	48	37	23	14	9	3	2	1
NOV 15	17	0.70	192	7.6	70	49	39	25	16	11	5	3	2
NOV 22	20	0.79	165	6.5	65	55	45	30	20	13	5	3	2
NOV 29	19	0.78	283	11.2	68	48	38	26	17	13	7	4	3
DEC 5	17	0.69	121	4.8	71	52	40	25	15	10	4	3	1
DEC 12	17	0.70	102	4.1	67	50	40	26	16	11	4	3	1
DEC 19	10	0.41	63	2.5	61	40	28	14	7	4	2	1	1
DEC 26	12	0.48	68	2.7	66	49	35	16	8	4	1	1	1
JAN 3	10	0.43	73	2.9	67	46	32	14	6	3	1	1	1
JAN 10	13	0.52	94	3.7	70	50	37	19	10	5	2	1	1
JAN 17	14	0.56	128	5.0	68	48	35	20	12	7	3	2	1
JAN 24	8	0.32	53	2.1	69	38	23	9	4	2	1	1	1
JAN 31	14	0.56	95	3.8	75	56	40	20	10	5	2	1	1
FEB 7	15	0.62	85	3.4	70	54	41	23	13	7	3	1	1
FEB 14	15	0.60	75	3.0	79	58	43	21	11	5	2	1	1
FEB 21	16	0.65	145	5.7	69	54	41	24	14	9	3	2	1

Source: Dugas (1983)

Table 25
Major Flood Events at Waco Reservoir

Date	Peak Flow at Reservoir Site (cfs)
September 1936	100,000
January 1938	43,100
April 1942	72,000
September 1942	39,000
May 1944	73,200
April 1945	153,800
February 1948	21,900
May 1952	19,200
April 1957	134,400
May 1968	40,000
April 1977	22,710
June 1981	27,170

times larger than for the same magnitude flood during the winter.

From a conservation perspective, raising the top of conservation pool from early April through the summer to capture additional April and May runoff for use during the summer would appear reasonable. However, a distinct flood season is difficult to precisely identify. April, May, September, and October are particularly critical months, but flooding could occur anytime during the year. Flood damage susceptibility is highest during the summer when water supply capacity is most needed.

The simulation and firm yield analyses discussed in the previous chapter were extended to include seasonal rule curve operating policies. The computations were accomplished with the HEC-5 computer program using the previously described input data. HEC-5 allows the top of conservation pool elevation to vary monthly. Operating policies consist of a specified average annual draft, which varies monthly in accordance with Table 10, and a top of conservation pool elevation, which may vary seasonally or remain constant. Alternative operating policies were simulated using historical period of record monthly streamflow data and average monthly evaporation rates. The alternative operating policies addressed in the following discussion includes the present top of conservation pool level of 455 feet as well as seasonal and permanent pool raises to elevations 462, 465, and 475 feet.

Firm yields for seasonally varying and constant operating policies are compared in Table 26. Constant top of conservation pool elevations of 455, 462, 465, and 475 feet result in firm yields of 81 cfs, 102 cfs, 108 cfs, and 127 cfs, respectively. Firm yields are also tabulated for seasonal rule curve operating policies in which the top of conservation pool is set at elevation 455 feet from November through March and raised to a higher level during April through October. Raising the top of conservation pool during April through October to elevations 462, 465, and 475 feet result in firm yields of 102 cfs, 108 cfs, and 126 cfs, respectively. Thus, either identically or practically the same increase in firm yield can be achieved by raising the top of conservation pool seasonally or permanently throughout the year.

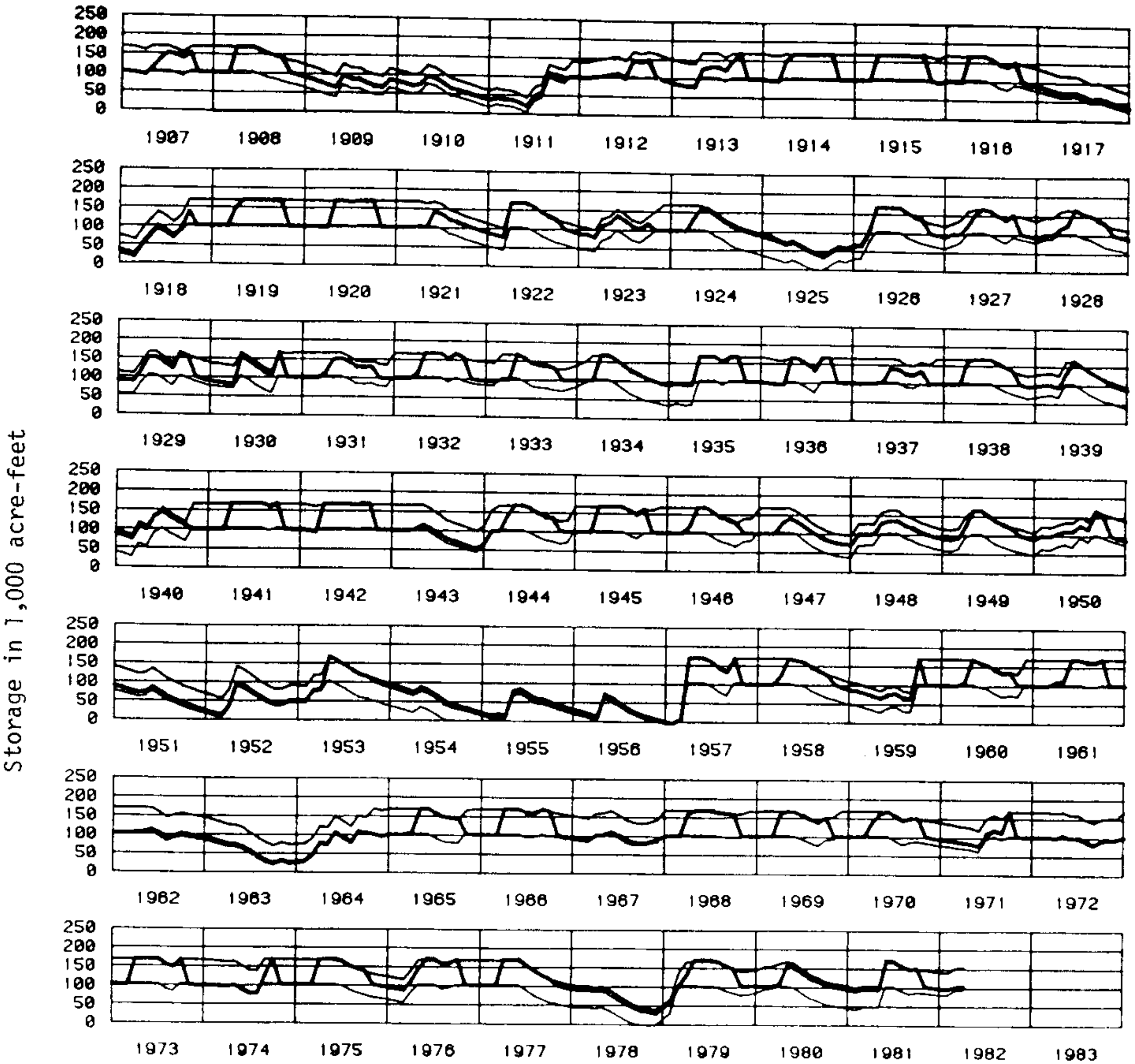
In computing firm yield based on historical streamflow records, the value obtained for firm yield is controlled by a critical drought period. Figures 17 and 18 compare the 455/465-feet seasonal rule curve with constant top of conservation pool elevations of 455 feet and 465 feet. An average draft of 102 cfs, varying monthly in accordance with Table 10, was included in each of

Table 26
Firm Yield for Constant and Seasonal Rule Curves

Type of Operation	Top of Conservation Pool (feet) Nov-Mar	Top of Conservation Pool (feet) Apr-Oct	Firm Yield (cfs)
constant	455	455	81
constant	462	462	102
constant	465	465	108
constant	475	475	127
seasonal	455	462	102
seasonal	455	465	108
seasonal	455	475	126

Table 27
Conservation Storage Capacities

Top of Conservation Pool Elevation (feet msl)	Conservation Storage Capacity (acre-feet)
455	104,100
462	148,000
465	172,000
475	270,000



Simulations for draft of 108 cfs (65.9 mgd) and alternative top of conservation pool elevations of 455 feet, 455/465 seasonal rule curve, and 465 feet.

Figure 17

Reservoir Simulation

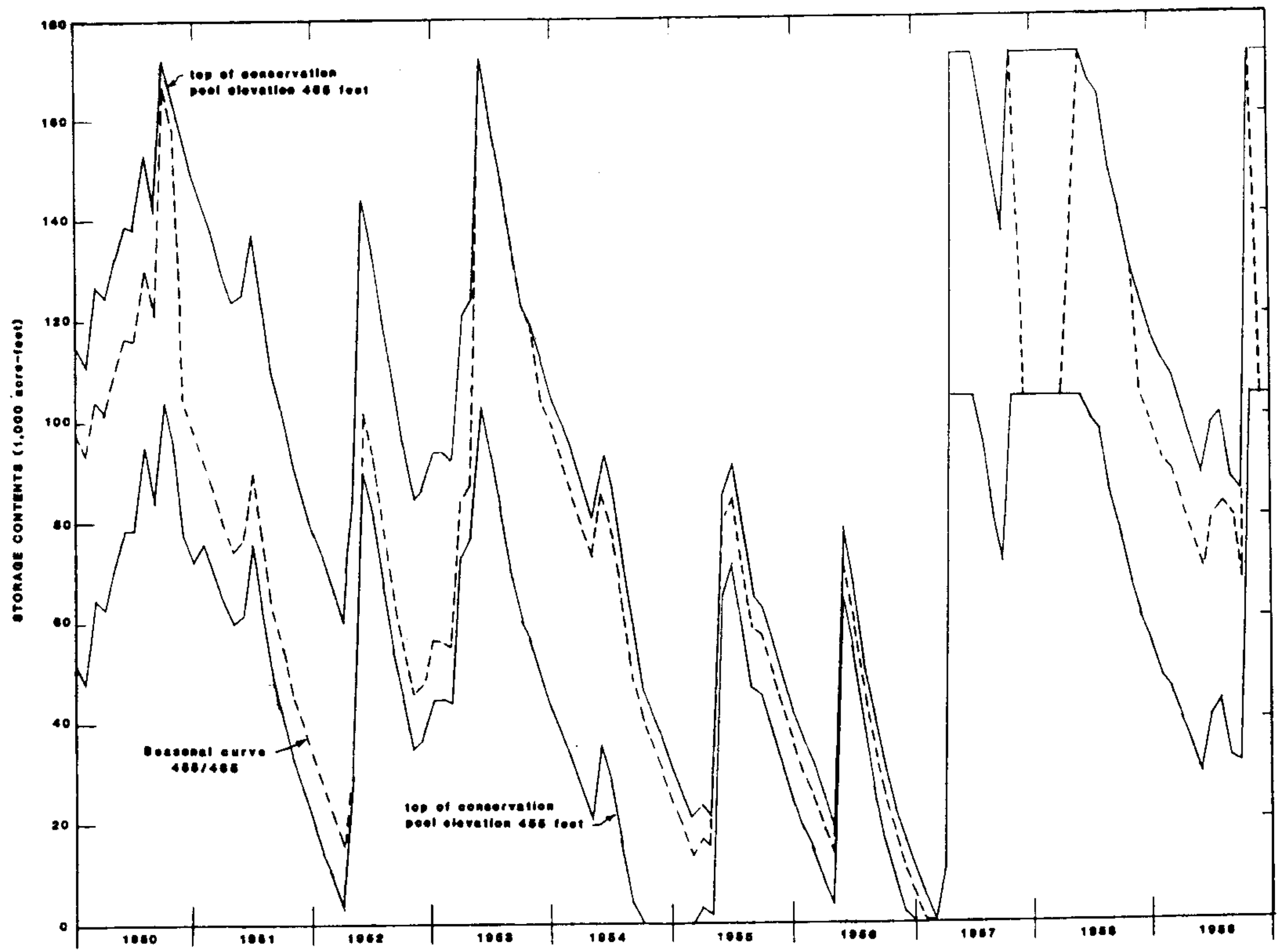


Figure 18 - Reservoir Simulation

the three simulations. The 108 cfs average draft is equal to the firm yield of both the 465-feet and 455/465-feet operating policies. Figure 17 is a plot of reservoir storage levels versus time computed using streamflow data for the entire period of record. A more detailed plot of storage levels during a series of years encompassing the critical period is presented in Figure 18.

As shown graphically in Figure 18, with either the permanent 465-feet or seasonal 455/465-feet operating policies, the reservoir is full, with 172,500 acre-feet of water in storage, at the end of May 1953. The 465-feet and 455/465 feet operating policies result in spillages of 1,026 acre-feet and 438 acre-feet, respectively, during the month of May. The critical period drawdown begins in June 1953 with the reservoir empty in February 1957 and full again in April 1957. In November 1953, the seasonal rule curve necessitated spilling 143 acre-feet to lower the water surface to elevation 455 feet. Thus, the reservoir storage levels are different for the 465-feet and 455/465-feet operating policies during the critical drawdown period. However, the difference is so small that the computed firm yields are the same. The reservoir is essentially refilled during the one month of April 1957, with a little refilling during March.

Storage levels resulting from operating at a permanent top of conservation pool elevation of 455 feet and draft of 108 cfs are also shown in Figures 17 and 18. The reservoir failed to meet demands due to being empty for several months in 1925, 1954, 1955, 1957, and 1978.

Raising the top of conservation pool from elevation 455 feet to 462 feet seasonally or permanently results in precisely identical increases in firm yield. The seasonal rule curve firm yield is identical to the permanent reallocation as long as the pool is raised no later than early May and lowered no earlier than late September. Simulations of the 455/462-feet and 462-feet alternatives using a draft of 102 cfs results in the same May 1953 to April 1957 critical period discussed in the previous paragraph. With either the 462-feet or 455/462-feet operating policy, the reservoir is full at the end of May 1953 with a storage of 148,000 acre-feet. The 462-feet and 455/462-feet policies result in spillages of 1,298 and 979 acre-feet, respectively, in May.

The critical period drawdown begins in June 1953 with the reservoir being essentially empty in February 1957 and full again in April 1957. The storage level drops below elevation 455 feet (104,100 acre-feet) during September 1953 and does not reach this level again until April 1957. The reservoir is essentially refilled during the one month of April 1957, with a little refilling during March. From May 1953 to past April 1957, the reservoir storage levels are identical for either the seasonal rule curve or permanent reallocation. Thus, the firm yields are identical.

A similar comparison of a 455/475-feet rule curve operation with a 475-feet permanent reallocation indicates the firm yields are almost the same. The seasonal rule curve consisted of raising the top of conservation pool to 475 feet from April through October. A draft of 127 cfs was used in simulating the two alternative operating policies. The permanent 475-feet policy results in a full reservoir (270,000 acre-feet) in June 1946 which begins to empty in July and does not fill again until April 1957. The reservoir is essentially empty in February 1957. For the 455/475 seasonal policy, the reservoir is full in July 1945, begins to empty in August and is full again in April 1957. The reservoir is empty in March 1952, February and April 1955, and October 1956 through February 1957. The 455/475 seasonal policy also results in the reservoir being empty in June 1911. Thus, with a 127 cfs draft, a permanent 475 feet top of conservation pool results in the reservoir being essentially empty during one month, while the seasonal rule curve results in the reservoir emptying several times during the hypothetical period of record simulation. However, the computed firm yields are practically the same.

Although seasonal rule curve operations can achieve essentially the same increases in firm yield as corresponding permanent reallocations, pool levels will tend to be lower under noncritical or more normal conditions of rainfall and streamflow. Table 28 shows the frequency of minimum annual storage levels equalling or falling below given levels. The data are based on determining the minimum storage level in each year of a period of record simulation and ranking the resulting annual data series. With a draft of 102 cfs, which is the firm yield for 462-feet and 455/462-feet operating policies, the seasonal rule curve results in storage levels approaching empty as frequently as the permanent 462-feet policy. However, less severe drawdowns occur more frequently with the seasonal rule curve. Similar results are indicated for the 455 versus 455/465 and 455 versus 455/475 analyses. The last group of simulations tabulated consists of combining a draft of 81 cfs, which equals the firm yield corresponding to a constant top of conservation pool elevation

Table 28
Minimum Annual Storage Frequency

Top of Conservation Pool (feet) Nov-Mar	Apr-Oct	Frequency (in percent) of Minimum Annual Storage Equalling or Falling Below the Following Levels (in 1,000 acre-feet) 0	10	20	30	40	50	100	150	200
					Draft = 102 cfs					
455	455	5	10	13	18	27	40	96	100	100
455	462	1	3	5	8	16	18	75	100	100
462	462	1	3	4	7	9	11	50	99	100
					Draft = 108 cfs					
455	465	7	13	17	22	30	43	97	100	100
455	465	1	4	8	12	16	19	75	100	100
465	465	1	2	3	5	7	9	29	82	100
					Draft = 127 cfs					
455	455	14	18	23	30	43	52	97	100	100
455	475	7	10	15	19	21	23	78	100	100
475	475	1	2	3	3	3	4	11	23	57
					Draft = 81 cfs					
455	455	1	3	5	8	13	20	93	100	100
462	462	0	0	0	0	3	5	37	99	100
465	465	0	0	0	0	0	1	17	70	100
475	475	0	0	0	0	0	0	0	8	26
455	462	0	0	0	0	4	9	65	100	100
455	465	0	0	0	0	4	9	65	100	100
455	475	0	0	0	0	4	9	65	100	100

of 455 feet, with the various alternative operating policies previously considered. The seasonal rule curve, in all cases, consists of raising the top of conservation pool from April through October.

Discussion

Lake Waco is considered typical of many of the multiple purpose reservoirs in Texas in regard to the potential for seasonal rule curve operation. To be effective from a conservation perspective, the seasonal rule curve must capture the May runoff for use during July and August. Capturing April runoff and keeping the pool raised through September and October would also enhance the conservation use of the reservoir. However, the likelihood of flooding is highest during April, May, September, and October. The rule curve could be raised during the month of May, gradually from the winter level in early May to the summer level by late May or possibly June. Most of the flood control capacity would be available most of May since some time is required to fill the increased conservation level unless a flood does occur. The reservoir could then be lowered in early September in anticipation of possible extreme flood events in September or October. The extra summer conservation capacity provided by the rule curve would usually be largely depleted of water by the end of summer anyway. The difficulty with this approach is that flood damage susceptibility to agricultural crops, which dominate the discharge versus damage relationships, is highest during May through July when the conservation capacity is most needed. Although flooding appears to be most likely in spring and fall, summer floods are still a significant concern. Seasonal rule curve operation is an effective means for increasing water supply yields, relative to permanent storage reallocation. However, tradeoffs between flood control and water supply benefits are still a major concern.

CHAPTER 8
SUMMARY AND CONCLUSIONS

Under present and projected future conditions of development and water use in the state, expanded management strategies and analysis capabilities are very important for preparing for the next severe drought, while, at the same time, optimizing the present beneficial use of limited storage capacity for the various purposes. Rapid population and economic growth and depleting groundwater reserves are resulting in ever increasing demands being placed upon surface water resources. Since public needs and objectives and numerous factors affecting reservoir operation change over time, operating procedures should be periodically reevaluated and modified whenever changing conditions so warrant.

Comprehensive integration of water management strategies could be a major future direction for optimizing the beneficial use of limited resources. This could include improved multireservoir system operation, integration of demand management with reservoir operation, and conjunctive surface and groundwater management. Systems analysis and hydrologic modeling can be expected to play an even greater role in the future in providing a quantitative basis for reservoir operation decisions.

In addition to examining reservoir operation in Texas in general and capabilities for analyzing reservoir operations in general, the present investigation focused on the particular management strategy of reallocating storage capacity between flood control and conservation purposes in response to changing conditions. Converting a portion of the storage capacity of a reservoir from one purpose to another is concluded to be a viable management strategy for optimizing the beneficial use of existing reservoirs under changing conditions. Although given relatively little consideration in the past, storage reallocations will likely be proposed more and more frequently in the future as demands on limited resources increase.

Seasonal rule curve operation may be a viable alternative to a permanent storage reallocation under certain circumstances. Many factors affecting reservoir operation are seasonal in nature. In the case study, firm yield was found to be significantly increased, relative to a permanent reallocation, by raising the top of conservation pool for only a portion of the year. However, the seasonal nature of flood threat and damage susceptibility results in a significant overlap between the time periods when flood control and water supply capacity are most needed. Consequently, seasonal rule curve operation still requires tradeoffs between water supply and flood control benefits.

A hydrologic and economic evaluation procedure was developed for analyzing proposed reallocations of storage capacity between flood control and municipal and industrial water supply. The generalized procedure consists of evaluating the risks and consequences of failing to prevent flooding and failing to meet water demands, associated with alternative allocations of storage capacity between flood control and water supply. Unlike traditional practices based on firm yield, water supply is treated analogously with flood control with risks and consequences being quantified. The major thrust of the proposed procedure is the estimation of average annual economic losses associated with alternative storage allocations. Average annual flood losses are computed using the damage-frequency method. Average annual water supply losses are estimated by developing a water shortage versus loss function which is then applied to water shortages computed by a hydrologic simulation. The water shortage versus loss function reflects emergency demand management and supply augmentation measures. Average annual water supply losses are estimated for a given demand level. Long-term demand management strategies are reflected in the water demand projections.

Reallocation of storage capacity involves complex institutional, financial, legal, political, and public opinion considerations as well as the hydrologic and economic factors addressed by the proposed procedure. The proposed procedure necessarily involves approximations and modeling uncertainties. The procedure is intended to provide an understanding of system performance and meaningful quantitative information for use in the decision-making process but not stringent criteria to be rigidly followed. The proposed procedure supplements and incorporates, rather than replaces, traditional methods. However, the economic evaluation provides meaningful additional information which should be very valuable as decisions regarding allocation of limited resources become more difficult.

The hydrologic and economic evaluation procedure was developed specifically for and applied to the analysis of proposed storage reallocations. However, the procedure could be adapted to a broad range of applications. The research demonstrated that risks and economic consequences of water shortages can be meaningfully quantified. Average annual losses due to failing to meet water demands can be estimated as accurately as the more traditional estimates of average annual flood damages. Economic evaluation capabilities will become increasingly more important in water supply planning and management as supply augmentation and demand management strategies are comprehensively integrated to meet ever increasing needs.

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